Drained Residual Shear and Interface Strength of Soils at Low Effective Normal Stress

by

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Drained Residual Shear and Interface Strength of Soils at Low Effective Normal Stress

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Dedication

This thesis is dedicated to my family who always love me

Hunoh Bae, Gabrye Lee, Seonghee Bae, Seonghoon Bae

To my friends who always trust me and make me smile

…

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Abstract

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The drained residual shear strength at the interface between soils and solid materials can be of importance in evaluating the stability of geotechnical structures. Drained residual shear tests have been performed at relatively high effective normal stress levels, over 50 kPa. These effective normal stresses are relevant for many field applications and manageable in typical laboratory shear testing. However, there are field applications, such as offshore pipelines where the effective normal stresses can be below 50 kPa. There are two significant challenges in measuring the drained shear strength at low effective normal stresses: (1) a small amount of friction in a test device can affect the results; (2) small shear rates may be required to achieve drained conditions at the soils. A

tilt table test method has been developed to overcome these challenges. The objective of this work is to measure the drained residual shear and interface strength of soils at low effective normal stresses so as to provide logical explanations of the effect of various parameters. These parameters include soil index properties, clay content, clay mineralogy, stress history, and loading rate together with the effective normal stress levels.

The total 74 tilt table tests are performed to measure the drained residual shear and interface strength of marine clays and sand-kaolinite mixtures. The following conclusions can be drawn based on the test results.

1. The drained residual shear strength both for the interface and for the soils is not affected by the over-consolidation ratio.
2. The drained residual shear strengths for the interfaces are all less than the drained residual shear strengths of soils. The drained residual strength of interface depends on the roughness of interface, clay mineralogy.
3. The empirical correlations and shear test results at higher effective normal stresses cannot be extrapolated to lower effective normal stresses.
4. Clay mineralogy and clay contents together with the magnitude of effective normal stress are the most important factors to estimate the drained residual shear strength of cohesive soils.
5. Cohesionless soils exhibit a constant residual secant friction angle regardless of effective normal stress levels.

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Chapter 1 Introduction

1.1 Background Information

The drained residual shear strength at the interface between soils and solid materials can be of importance in evaluating the stability of geotechnical structures, such as submarine pipelines, anchor rods, earth reinforcement and offshore friction piles. Since the soil-interface strength is generally different from the drained residual strength of soils and strongly related to the interface roughness and the properties of the soils, special attention should be paid to the influence of the presence of interface in changing the failure mechanism. Drained residual shear tests have been performed for many years at relatively high effective normal stress levels, say greater than 50 kPa. These effective normal stresses are relevant for many field applications and manageable in typical laboratory shear testing, such as direct shear tests and triaxial shear tests. However, there are field applications where the effective normal stresses are below 50 kPa. For instance, offshore pipelines are subjected to thermal expansion and contraction due to temperature changes during carrying product along the sea floor and the effective normal stresses acting on the interface between a pipeline and the soil is generally ranging from 1 to 5 kPa (Najjar, Gilbert et al. 2007). Since the residual shear strength parameters vary with applied effective normal stress, careful attention is required to define the complete residual failure envelope at low effective normal stresses.

There are two significant challenges in measuring the drained shear strength at low effective normal stresses: (1) a small amount of friction in a test device can affect the results; (2) small shear rates may be required to achieve drained conditions at the soils. A tilt table test method has been developed to overcome these challenges. It eliminates the need for a mechanical loading system by using gravity to apply the normal and shear stresses to the soil specimens and a thin film of soil, 2 to 3 mm in thickness, to provide for efficient drainage of pore water pressures, internally and against interfaces. This study is focused on understanding of the tilt table method and the drained residual shear and interface strength of soils at low effective normal stresses.

1.2 Research Objectives and Scope

The first objective of this study is to measure the drained residual shear strength at the interface between two pipeline coatings having different roughnesses and marine clays from three different locations at an offshore project site. Results are compared with the drained residual strength of soils. These results will be used to estimate the soil-pipe resistance to aid in assessing the overall stability of the pipeline.

The second objective is to provide a better understanding of the effect of various parameters, such as soil index properties, clay content and clay mineralogy on residual shear strength and the interface strength. The objective is extended to investigate the effect of stress history, loading rate and the magnitude of effective normal stresses on the drained residual shear strength of soils and at the interface. Test results are developed using kaolinite, sand and kaolinite-sand mixtures.

Seventy-four tests with marine clays and laboratory-prepared soil mixtures with different proportion of clays and different interfaces are described and analyzed to meet these objectives.

1.3 Structure of Thesis

The introduction, including objective and scope of this study is presented in Chapter 1. A review of previously published work on measuring drained residual shear and interface strength of soil is summarized and discussed in Chapter 2. Chapter 3 provides a description of the tilt table apparatus. A description of the test procedure is described in detail in Chapter 4. The test program, including a description of the soils and interfaces tested is addressed in Chapter 5. The test results are presented and analyzed in Chapter 6. The test results are compared with the previous studies and discussed in Chapter 7. Finally, a summary of the major conclusions obtained from this study is presented in Chapter 8. Raw data of all tests and additional test results are provided in Appendix.

Chapter 2 Literature Review

In this chapter, previously published work on measuring drained residual shear and interface strength of soils is summarized. This previously published work focuses on the characteristics of residual strength of soils and correlation between the measured residual strength and index properties of soils at relatively high effective normal stress, greater than 50 kPa. The direct shear box and the torsional ring shear device are the most commonly used methods for measuring both the drained residual shear strength of soils and at interface with soils.

2.1 Drained Residual Shear Strength of Soils and at Interface

(Lupini, Skinner et al. 1981) had performed tests in the ring shear apparatus using sand, bentonite, and sand-bentonite mixtures to measure the drained residual strength of soils with a different clay minerals and different fractions of clay-size particles. Index properties and tests results are summarized in Table 2.1. The residual strength for these tests was defined as the shear stress at the total displacement of about 1,000 mm.

Table 2.1 Ring Shear Tests on Sand-Bentonite Mixtures (Lupini, Skinner et al. 1981)

Test no.	Soil	Clay fraction (%)	Liquid limit (%)	Plasticity index (%)	Normal effective stress (kPa)	Peak secant friction anlge	Residual secant friction anlge	Water content at failure (%)
1	Sand	0	N/A	N/A	352	34	30	24
					177	35	30	
					352	32	30	
2	85%sand/15% bentonite	13	38	17	352	32	30	23.7
					177	38	30	
3	70%sand/30% bentonite	26	56	36	352	26	24	29.1
					177	29	22	
4	55%sand/45% bentonite	40	80	57	352	23	17	40.2
					177	17	14	
5	40%sand/60% bentonite	53	114	86	352	19	7	52.6
					177	4	4	
6	25%sand/75% bentonite	66	140	104	352	21	6	62.8
					177	4	4	
7	Bentonite	88	184	136	703	Consoilidation		74.7
					352	21	6	
					177	5	5	
					352	6	6	

Water content at failure was measured at effective normal stress of 177 kPa.

(Tika-Vassilikos 1991) used the ring shear apparatus to measure the interface strength between London Clay and stainless steel. The index properties of the clay were a liquid limit of 71%, a plastic limit of 26% and a clay fraction (percentage of particles less than 2μm by dry weight) of 53%. The specimens were consolidated to a normal stress of 967 kPa and then swelled back to 484 kPa. The estimated water content of the specimens at the end of the swelling stage was 30.5%. The initial thicknesses of the specimens were 19 to 12.5 mm. The specimens were sheared initially with fast rates and then slow drained shearing was carried out. The residual condition was mobilized at displacements of about 30mm and measured drained residual friction angle of soil and interface were 11° and 8.8°, respectively. Measured roughness of the interface indicated a center-line-average roughness (CLA*) of 8.4 μm.

(Lehane and Jardine 1992) carried out a series of ring shear experiments to measure the residual shear strength of Bothkennar clay and the interface between stainless steel and Bothkennar clay. The clay had a liquid limit of 80%, a plastic limit of 32% and a clay fraction of 35%. The CLA value of interface was about 8.5μm. In order to achieve drained conditions, the clay was sheared at a rate of 0.008 mm/min for a displacement of about 50 mm. The measured residual friction angle was about 32° and 30° in soil and at the interface, respectively, under an effective normal stress of 50 kPa. Visual inspection of the specimens after testing indicated that failure occurred within the soil specimen in both tests.

(Tsubakihara, Kishida et al. 1993) tested the shear strength between cohesive soils and mild steel using a direct simple shear apparatus. Kawasaki marine clay, having a plastic index of 48%, a liquid limit of 86% and a clay fraction of 60% was used in this study. All tests were conducted under an effective normal stress of 294 kPa and sheared at a constant speed of 0.03 mm/min to achieve drained condition. The thicknesses of the specimens were about 14 mm and the residual shear strength was defined at the total

* The CLA is the arithmetical mean of the areas of all profile values of the roughness profile

displacement of 15 mm. The measured residual secant friction angle varied with the roughness of the interface. As the roughness of the interface increased from 3 to 30μm, the residual secant friction angle increased from 20° to 28°, while the secant friction angle of soil was about 27°. For the steel roughness more than 10μm, interface sliding was not observed due to shear failure within the soil. They also studied the shear between soil mixtures, having a ratio of sand to clay of 0.2, 0.4, 0.6, 0.8, and 1.0, and steel, having roughness ranging from 3 to 80 μm. They concluded that the critical value of interface roughness, which is the boundary value above which internal failure occurs within the soil, increased for a soil with a higher sand fraction.

(Stark and Eid 1994) used a torsional ring shear apparatus to measure the drained residual shear strength of 32 clays and shales, having index properties as presented in Table 2.2. The specimens were sheared at effective normal stresses between 50 and 700 kPa and an over-consolidation ratio ranging from 14 to 1, respectively, and at a drained displacement rate of 0.018 mm/min. The displacement required to achieve a residual condition was approximately between 10 and 20 mm.

Table 2.2 Clay and Shale Samples Used in Ring Shear Tests (Stark and Eid 1994)

Soil number (1)	Clay and shale samples (2)	Clay and shale locations (3)	Initial water content (%) (4)	Specific unit weight (kN/m^3) (5)	Liquid limit (6)	Plastic limit (7)	Clay-size fraction (%) (8)	Activity (PI/CF) (9)
1	Glacial Till	Urbana, Ill.	8.4	16.1	24	16	18	0.44
2	Loess	Vicksburg, Miss.	14.0	16.5	28	18	10	1.00
3	Bootlegger Cove clay	Anchorage, Alaska	34.8	18.6	35	18	44	0.39
4	Duck Creek shale[a]	Fulton, Ill.	5.3	24.0	37	25	19	0.63
5	Chinle (red) shale[a]	Holbrook, Ariz.	10.9	22.7	39	20	43	0.44
6	Colorado shale[a]	Montana, Mont.	5.6	21.2	46	25	73	0.29
7	Panoche mudstone	San Francisco; Calif.	14.2	19.6	47	27	41	0.49
8	Four Fathom shale[a]	Durham, England	3.3	25.1	50	24	33	0.79
9	Mancos shale	Price, Utah	4.9	24.5	52	20	63	0.51
10	Panoche shale	San Francisco, Calif.	12.0	20.2	53	29	50	0.48
11	Comanche shale[a]	Proctor Dam, Tex.	11.5	23.1	62	32	68	0.44
12	Bearpaw shale[a]	Billings, Mont.	15.7	21.8	68	24	51	0.86
13	Slide debris	San Francisco, Calif.	18.1	19.6	69	22	56	0.84
14	Bay mud	San Francisco, Calif.	73.0	15.0	76	41	16	2.19
15	Patapsco shale[a]	Washington, D.C.	21.6	20.7	77	25	59	0.88
16	Pierre shale[a]	Limon, Colo.	24.3	20.1	82	30	42	1.24
17	Santiago claystone	San Diego, Calif.	20.7	19.6	89	44	57	0.79
18	Lower Pepper shale	Waco Dam, Tex.	21.0	20.3	94	26	77	0.88
19	Altamira bentonitic tuff	Portuguese Bend, Calif.	62.0	17.5	98	37	68	0.90
20	Brown London clay	Bradwell, England	33.0	18.9	101	35	66	1.00
21	Cucaracha shale[a]	Panama Canal	18.4	20.7	111	42	63	1.10
22	Otay bentonitic shale	San Diego, Calif.	27.0	17.6	112	53	73	0.81
23	Denver shale[a]	Denver, Colo.	30.5	18.7	121	37	67	1.25
24	Bearpaw shale[a]	Saskatchewan, Canada	27.3	19.0	128	27	43	2.35
25	Oahe firm shale	Oahe Dam, S.Dak.	27.6	20.1	138	41	78	1.24
26	Claggett shale[a]	Benton, Mont.	11.7	22.7	157	31	71	1.78
27	Taylor shale[a]	San Antonio, Tex.	35.2	18.0	170	39	72	1.82
28	Pierre shale[a]	Reliance, S.Dak.	42.8	17.7	184	55	84	1.54
29	Oahe bentonitic shale	Oahe Dam, S.Dak.	35.4	18.9	192	47	65	2.23
30	Panoche clay gouge	San Francisco, Calif.	34.8	21.8	219	56	72	2.26
31	Lea Park bentonitic shale	Saskatchewan, Canada	36.0	17.3	253	48	65	3.15
32	Bearpaw shale[a]	Ft. Peck Dam, Mont.	15.8	21.8	288	44	88	2.77

[a]Index properties from Mesri and Cepeda-Diaz (1986).

Figure 2.1 presents all test results as a residual secant friction angle and a correlation of drained residual friction angle and soil index properties, at effective normal stress of 100,400, and 700 kPa. These results also present the nonlinearity of the drained residual failure envelope, meaning that the secant friction angle decreases with increasing effective normal stress. The measured residual secant friction angle ranged from 6˚ to 32˚, and varied with the clay-size fraction and liquid limit.

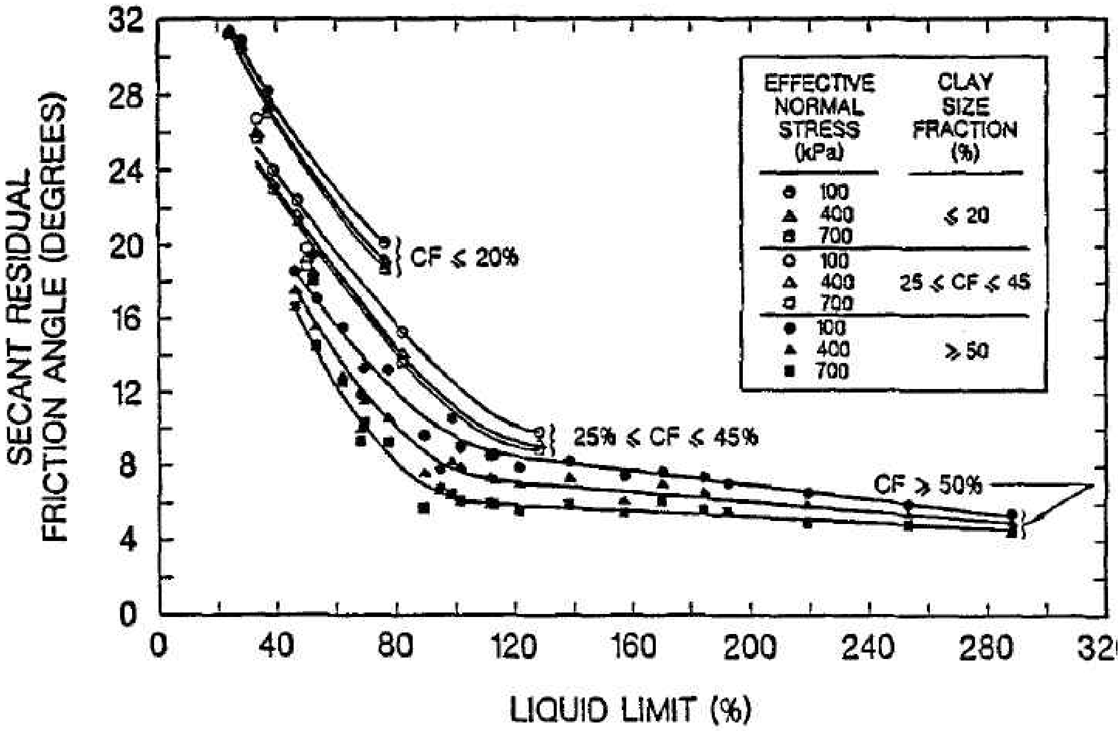


Figure 2.1 Relationship between Drained Residual Friction Angle and Liquid Limit (Stark and Eid 1994)

(Lemos and Vaughan 2000) studied the shear resistance between clays of varying plasticity and interfaces of varying roughness using a ring shear apparatus. Ring shear tests on six different clays of low plasticity were conducted against three different types of interfaces, glass, mild steel and stainless steel. A residual strength was reached after a few tens of millimeters displacement. All tests were conducted at effective normal stresses ranging between 50 to 400 kPa. Residual secant friction angles ranged from 15° to 28° for the residual shear strength of the soils. As the interface roughness increased from 0.005 to 7µm, the measured residual interface efficiency, defined as the ratio of the residual interface shear strength to the residual shear strength of soil, increased from 40 to 90%. Direct reversal shear tests on kaolinite, having a plastic index of 36%, a liquid limit of 72% and a clay fraction of 82%, were also conducted to measure the residual shear strength of pure clay against a smooth interface with a mean CLA of 0.22μm. Samples were sheared at a rate of 0.0337 mm/min at an effective normal stress of 200 kPa. The measured residual secant friction angle was about 10° for the residual strength of the interface, while the soil-on-soil value was about 18° as obtained from Lupini *et al* (1981). The displacement needed to achieve the residual condition in soil against interface tests was about 10 mm, while that in soil tests was about 100 mm.

Taylor and Leps (1938) performed two direct shear tests on Ottawa sand at the same normal stress but different density, one sample was dense and another one was loose (Figure 2.2). At the beginning of shearing, dense sand exhibits the higher shear strength, but at large displacements the two sands shows the same shear strength. These tests suggest that the initial relative density of sands does not affect the residual strength that is of interest in the current study. In case of cohesive soils, the specimen preparation and stress history also are not expected to influence residual strength (Bishop, Green et al. 1971).

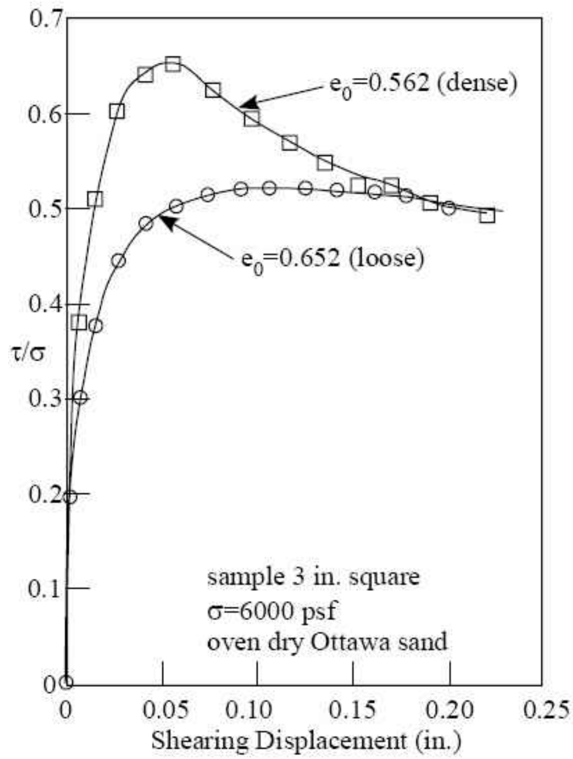


Figure 2.2 Direct Shear Tests on Dense and Loose Ottawa Sand (Taylor, Leps 1938)

2.2 Empirical Correlations for Drained Residual Shear Strength

Lupini *et al* (1981) conclude that “correlations between residual shear strength and soil index properties cannot be general”. However, for a certain soil type, reasonable correlations may be possible and these correlations may provide guidance to estimate the drained residual shear strength of soils. Many empirical correlations for drained residual shear strength have been described that are a function of liquid limit, clay fraction, plasticity index and effective normal stress. Most of them are based on one soil index property, as presented in Table 2.3.

Table 2.3 Comparison of Empirical Correlations for Drained Residual Shear Strength Using Field Case Histories (Stark and Eid 1994)

		Gardiner Dam (LL = 128; PI = 101; CF = 43)		Portuguese Bend (LL = 98; PI = 61; CF = 68)	
Soil index property (1)	Reference (2)	Residual friction angle (3)	Factor of safety (4)	Residual friction angle (5)	Factor of safety (6)
Liquid limit and clay fraction	Current study	9.8	1.02	6.9	1.04
Clay fraction	Skempton (1964)	18.0	1.91	12.3	1.67
	Borowicka (1965)	7.5	0.77	7.5	1.12
	Binnie et al. (1967)	15.2	1.61	11.5	1.58
	Blondeau and Josseaume (1976)	12.6	1.31	7.0	1.06
	Lupini et al. (1981)	13.4	1.41	4.9	0.81
	Skempton (1985)	—[a]	—[a]	11.1	1.53
	Collotta et al. (1989)	11.8	1.23	7.8	1.15
Plasticity index	Fleischer (1972)	9.1	0.94	9.1	1.30
	Voight (1973)	6.3	0.65	9.8	1.38
	Kanji (1974)	6.0	0.61	7.5	1.12
	Bucher (1975)	—[a]	—[a]	—[a]	—[a]
	Mitchell (1976)	10.9	1.13	11.6	1.59
	Seyček (1978)	7.8	0.81	9.6	1.36
	Vaughan et al. (1978)	—[a]	—[a]	—[a]	—[a]
	Lambe (1985)	—[a]	—[a]	—[a]	—[a]
	Clemente (1992)	11.6	1.21	11.6	1.59
Liquid limit	Haefeli (1951)	—[a]	—[a]	—[a]	—[a]
	Mitchell (1976)	13.0	1.36	13.0	1.75
	Mesri and Cepeda (1986)	9.4	0.97	8.0	1.18

[a]Not applicable.

Various correlations have been proposed for the drained residual shear strength and index properties as follows: high plasticity clays exhibit typically low residual friction angle (Early and Skempton 1972); the residual shearing angle decreases with increasing clay-size fraction (Lupini, Skinner et al. 1981); there is no satisfactory relationship between residual shear strength and plasticity index and clay mineralogy is the most important factor on residual shear strength (Kenney 1967); the residual shear strength is independent of initial soil structure and stress history (Bishop, Green et al. 1971); the residual shear strength is also independent of effective normal stress level when stresses in excess of 150 kPa are used (Townsend and Gilbert 1973; Townsend and Gilbert 1976), Since the clay fraction indicates quantity of clay particles smaller than 2μm and the liquid limit can represent the type of clay mineralogy, the drained residual strength is expected to decrease as the liquid limit, clay fraction and activity* increases (Stark and Eid 1994). Most previous studies presented here have indicated that the residual shear strength decreases with increasing clay content from that of a non-cohesive soil to that of a pure clay. When soils tested against a smooth, hard interface, partial sliding could occur at the interface, resulting in a lower residual shear strength than for soil alone (Skinner 1969).

Lupini *et al.* (1981) suggested that there are four possible failure modes of residual shear behavior, depending on inter-particle friction angle and particle shape, described as turbulent shear and sliding shear. When the clay particles predominate, the oriented clay particle could form a shear zone between the well-dispersed rotund particles of sand and shearing is predominantly by sliding of the oriented clay particles. This mode was called sliding shear and shearing resistance depended on inter-particle friction. When rotund particles predominate, shear is by rotation of the rotund particles, and shearing

* The ratio of the plasticity index to the clay-size fraction.

resistance is no longer controlled by inter-particle friction. This mechanism was called turbulent shear. The change in residual friction coefficient with increasing clay fraction and the differences in the trends of change with the magnitude of effective normal stress might be due to the existence of these two failure mechanisms.

When both types of shear occur simultaneously, this type of shear was described as transitional shear. Figure 2.3 illustrates the typical zone in which these types of mechanisms occur, as a function of clay fraction and granular void ratio. Granular void ratio is defined as the ratio of the volume of platy particles and water to the volume of rotund particles and S_1 is possible sliding shear when soil is failed against a smooth, hard interface (Lupini, Skinner et al. 1981).

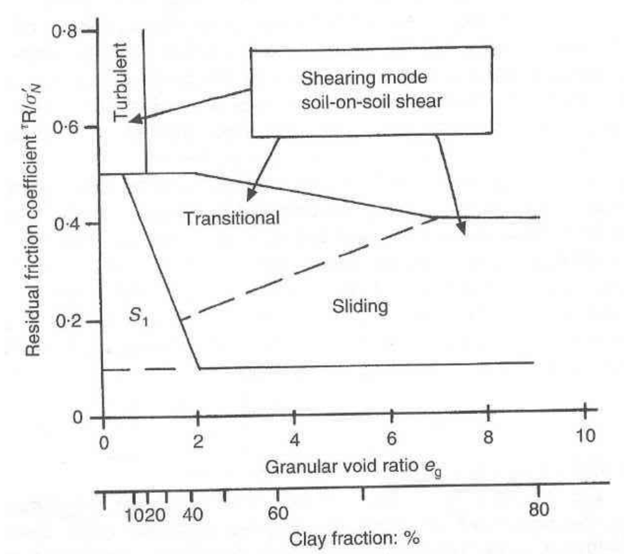


Figure 2.3 Residual Shear Mechanisms as a Function of Clay Fraction

(Lemos and Vaughan, 2000)

Lupini *et al.* (1981) presented data for the residual shear strength of cohesive soils, and following conclusions were drawn. For the turbulent behavior; (1) Soils exhibit a high residual strength, typically with residual friction angle greater than 25˚, and no particle orientation occurs so that the soil shows no brittleness for the first time of failure; (2) Increasing clay fraction leads to separation of the contacts between the rotund particles, results in a reduction in strength; (3) Turbulent mode could occur regardless of the clay fraction, if the inter-particle friction angle is enough high perhaps due to a high salt concentration in the pore fluid; (4) The residual friction angle depends primarily on the shape and packing of the rotund particles, not on the inter-particle friction.

For the sliding behavior; (1) Soils exhibit typically low shear strength along the strongly oriented clay particles; (2) The clay has a higher activity exhibits the lower residual friction angle, indicating that the residual friction angle depends primarily on clay mineralogy and inter-particle friction; (3) The clay exhibits brittleness for the first shearing due to the clay particle reorientation.

Note that these conclusions were based on the test results performed under the effective normal stresses higher than 100 kPa. Therefore, the change in shearing behavior with clay fraction at very low effective normal stresses may be different.

2.3 Review of Test Devices

Direct shear devices have been used to measure the friction angle between soil and solid material (Ling, Burke et al. 2002). However, a relatively large confining pressure is required to control the normal pressure precisely and it is difficult to obtain the residual condition in a direct shear device due to the limited travel distance.

The reversal direct shear test has been also used to measure the drained residual strength of soils, although it has the following limitation. The soil is sheared forward and then backward until constant shear strength is obtained. Therefore, there is no continuous shear displacement in the soil specimen in one direction, and thus reorientation of the clay particles to the direction of shear may not be obtained. Testing in these devices allows the application of limited shear displacements in one direction. The use of the ring shear apparatus can overcome this problem. The specimen can be sheared through an unlimited displacement continuously without having to stop and reverse the shearing movement.

The ring shear apparatus has been widely used to measure both peak and residual strength of soils (Bishop, Green et al. 1971). The specimen is 152 mm outside diameter, 102 mm inside diameter, and 19 mm in thickness. The modified torsional ring shear apparatus shears the soil specimen in one direction to large displacement, thus to be allowing clay particles to be oriented to the direction of shear and a residual condition to be developed (Stark and Eid 1994). In addition, the ring shear apparatus provides a constant cross section area of the shear surface during shearing process, while the direct shear test may need a correction for the shearing area with displacement. However, uniform stresses cannot be developed with the soil specimen until a residual condition is reached because the strains and displacements are 50% greater on the outside than on the inside for a given rotation.

2.4 Summary and Discussion

The test results presented above involve effective normal stresses that are one to two orders of magnitude higher than the stress levels of interest in this study. All studies to measure the drained residual shear strength of soils or soil-interfaces and to correlate that with soil properties using conventional test devices presented above involve normal stresses from 50 kPa to 1,000 kPa. Since the secant friction angles for the residual strengths decrease as the effective normal stress increases, it is not possible to extrapolate the empirical correlations for residual shear reported in the literature at effective normal stresses greater than 50 kPa to effective normal stresses on the order of 5 kPa.

The conventional test devices use a mechanical loading system to apply normal and shearing stresses to the soil specimen. At low effective normal stresses, the friction in the mechanical loading system can provide a significant error in the measured residual shear strength. Tilt table test methods have been used to overcome these limitations by using gravity to apply normal and shear stresses to the soil specimen (Pedersen, Olson et al. 2003).

Many of previous studies on soil-interface shear strength have been conducted on sands rather than clays. The interface shear strength of sand depends on the roughness of the interface materials, the size of the sand particles and effective normal stresses. As interface roughness increases, failure tends to occur within the sand. When the sand is sheared against a very rough surface, the sand interface shear strength equals that of the sand itself (Yoshimi and Kishida 1981; Uesugi and Kishida 1986a; Uesugi and Kishida 1986b). When clays are sheared against solid materials, the interface shear strength is generally less than the shear strength of the soil, and decreases with decreasing interface roughness. The displacement required to achieve the residual conditions is less when shearing clay against a solid material than it is when shearing clay against clay, and it also decreases with decreasing roughness of the interface. Visual observations of the failure surface between clay and solid material have indicated that much of it may involve shear of clay against clay.

Chapter 3 Test Apparatus

3.1 Introduction

The objective of this chapter is to describe the details of the tilt table test device.

3.2 Tilt Table Frame

The tilt table has an aluminum base plate that is 460 x 700 mm in area and is hinged to a steel frame. In order to apply the shear stress, a winch and gear are used to lift the free end of the base plate with designated surcharge weight. The maximum titling angle is about 45˚. The interface material is attached to the base plate with clamps. The tilt table frame is shown on Figure 3.1.

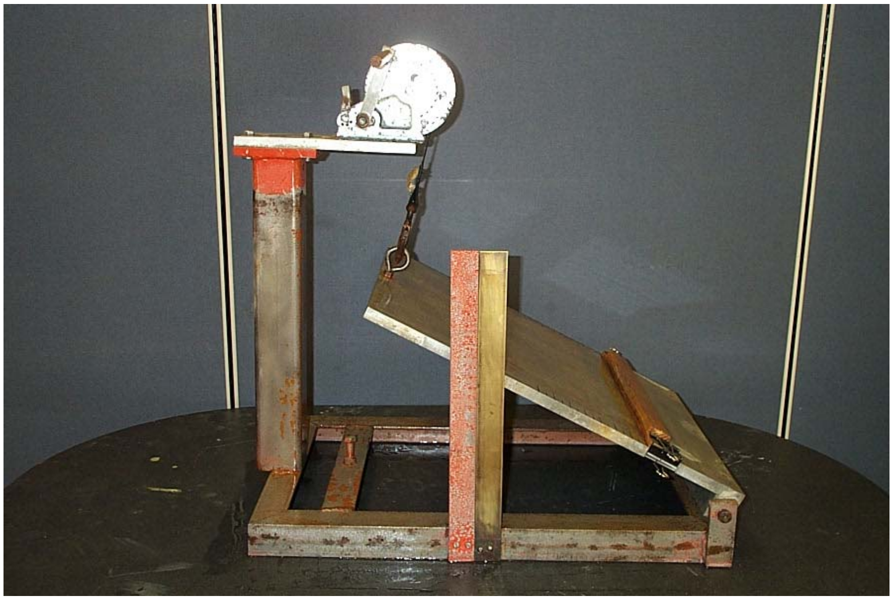


Figure 3.1 Tilt Table Frame

3.3 Loading Plates

In this study, an acrylic plate and a steel plate are fabricated as upper loading plates (Figure 3.2) and referred to in this study as D6A and D6S respectively. The steel rod of each loading plate is screwed into the upper loading plate to support the steel weight and adjust the center of gravity of the applied loads. The D6A has a thickness of about 20 mm and a diameter of 152 mm. The D6A applies a normal stress of 0.08 kPa. The D6S has a thickness of about 25.4 mm and a diameter of 152 mm. The D6S applies a normal stress of 0.7 kPa. For higher normal stresses, the D6S is placed on the soil specimen.

Both plates have a drainage material at the bottom of the plate to provide freely draining conditions at the top of the soil specimen. This drainage material must be rough enough to prevent the failure plane from creating between soil and drainage material and smooth enough to prevent the drainage material from protruding into the soil and affecting the results. In order to find a proper drainage material, different types of drainage layers coupled with different thickness of soil specimens were investigated. In some tests, a porous stone that was 152 mm in diameter and 10 mm in thickness was glued to the bottom of the plate to provide draining condition. However, it was too smooth to avoid failure between the soil and porous stone, especially under relatively low effective normal stress, less than 6 kPa. Using trial and error, a nonwoven, needle-punched geotextile was chosen for this study. This geotextile must be replaced after about ten of tests are conducted to eliminate the possibility that the clay particles intrude into the geotextile and reduce its drainage capacity.

(a) D6S (b) D6A

Figure 3.2 Upper Loading Plate (steel and acrylic)

For undrained shear tests, an impermeable textured geomembrane was used to prevent the drainage from the top of the specimen during shearing (Figure 3.3).

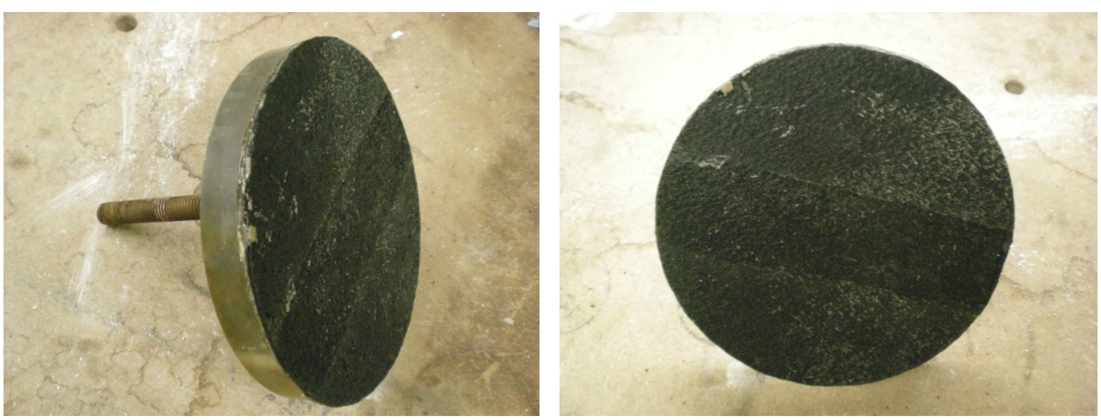


Figure 3.3 Upper Loading Plate with Impermeable Textured Geomembrane

3.4 Surcharge Weight

The types of surcharge weight are presented in Figure 3.4. A set of weights made specifically for one-dimensional consolidation tests is used in this study (Olson 1986). These weights are marked with the pressures that they can apply to a 2.5-inch diameter consolidation sample. Each steel block is designated by the pressure marked on them as shown in Figure 3.5. The actual pressures depend on the area of soil sample used in this study. The effective normal stress applied by each steel block is calculated in Table 3.1.

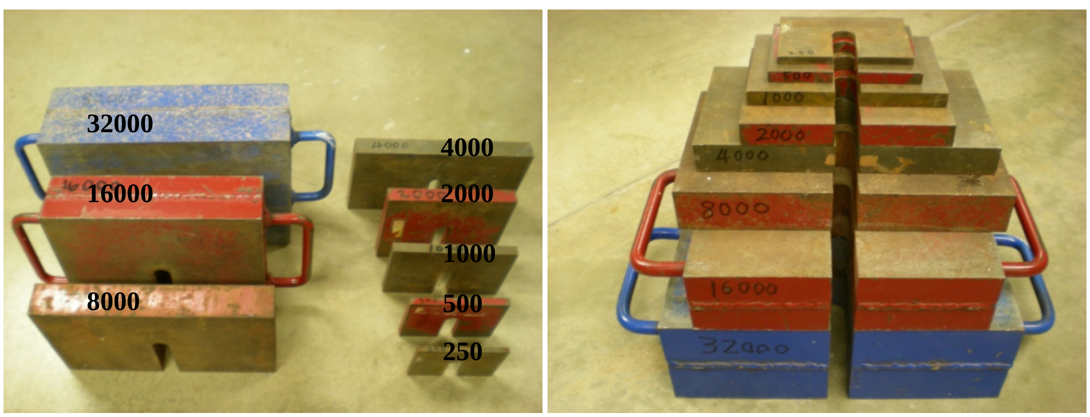


Figure 3.4 Surcharge Weight used in this study

Table 3.1 Pressure Applied by Each Steel Weight

Pressure label (#)	Normal Weight (lb)	Submerged pressure in water		
		P (kPa)	P (psi)	P (psf)
32000	99.2	21.07	3.06	439.93
16000	49.6	10.54	1.53	219.96
8000	24.8	5.27	0.76	109.98
4000	12.4	2.63	0.38	54.99
2000	6.2	1.29	0.19	27.03
1000	3.1	0.66	0.1	13.8
500	1.552	0.33	0.05	6.99
250	0.766	0.16	0.02	3.44

Chapter 4 Test Method

4.1 Introduction

The objective of this chapter is to describe the details of experiments that were performed in this study on marine clays, kaolinite, sand, and sand-kaolinite mixtures. The soil is spread on the interface material 2 to 3 mm in thickness and a static load is applied on the horizontally oriented interface to fully consolidate the soil under the applied normal stress. The base plate is then tilted to apply a shear stress at a slow enough rate to achieve drained conditions until failure occurs. For undrained tests, the tilt table is raised at a fast enough rate to achieve undrained conditions during shear. The tilt table is lifted until the upper loading plate slides down about 15 mm. The table is iteratively lowered and raised until the friction angle is a constant (i.e., more than two successive failure occurs at the same angle) so as to reach the residual conditions. The water contents are measured from the failure surface after testing. A schematic of the test method is shown Figure 4.1. The details of each procedure are described as follows.

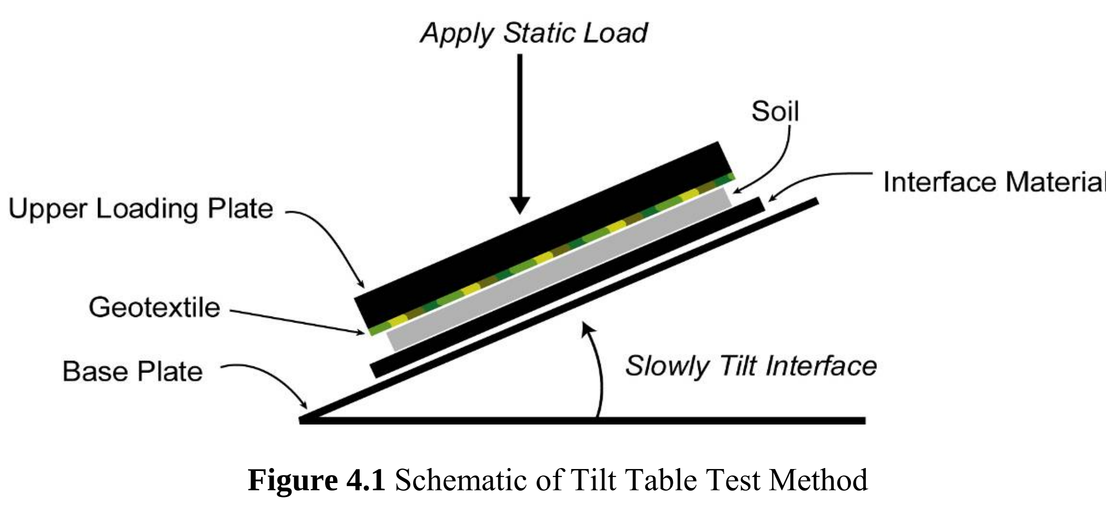


Figure 4.1 Schematic of Tilt Table Test Method

4.2 Specimen Preparation

Since the drained residual strength is of interest and the initial structure of the soil does not affect the drained residual strength (Bishop, Green et al. 1971), all of the soils are placed in the remolded state. This approach leads the structure of soil to near the fully softened state at the start of the test and can minimize the needed displacement to achieve the residual conditions. In case of sand, since the initial relative density does not affect the residual strength (Taylor, Leps 1938), the sands are placed on the interface as dense as possible. This approach provides more visible failure surface after the tests and leads to keep the specimen in initial shape under water.

The marine clays from three different locations at an offshore project site are prepared by mixing the soil thoroughly to ensure homogeneity and then transferred to sealable plastic bags to maintain the initial water content. The target water content is estimated by extrapolating the virgin consolidation curve ($e - \log\sigma_n$ curve) from one dimensional consolidation test data back to the low effective normal stress values used in the study. The target water content is usually near or above the liquid limit of soils. The soil is prepared to the target water content by adding salt water to keep the salinity of pore-water that already was present after measuring initial water content for each specimen from the sealable plastic bags.

The kaolinite and sand used for comparison purposes were provided in a dry state. The kaolinite is prepared to the target water content by adding tap water with 70 percent dry weight of kaolinite and mixing thoroughly to achieve homogeneity. The sand is fully saturated with tap water, and the water content after spreading on the interface is about 30 percent. For sand-kaolinite mixtures, the kaolinite is thoroughly mixed with sand in the dry powdered state in different proportions of dry weight of kaolinite ranging from 10 to 70 percent and saturated with a designated amount of tap water (70 percent of the dry weight of kaolinite) to achieve the target water content for soil mixtures.

Two wood frames are used so that the prepared soil specimens would be uniform in thickness. Each wood frame has a thickness of 2 and 3 mm respectively and 170 x 250 mm in area. In order to avoid trapped air during preparing the soil specimen, the soil specimens are spread out and kneaded in small amounts with the spatula. Using thin specimens on remolded soil has the following advantages: (1) the time required for consolidation and dissipating excess pore water pressures induced by shearing are minimized; (2) the need for lateral confinement is eliminated because the shearing surface is very long relative to the thickness of the soil at the boundaries. However, the thickness of the specimen needs to be large enough so that the drainage material at the bottom of the loading plate does not protrude into the soil and contact the interface at the top of the base plate. This effect will cause an increase in the measure shear resistance. Observation of the failure plane during and after the tests indicates that the thickness of 2 mm is large enough to prevent this effect for effective normal stresses less than 10kPa.

However, under the highest effective normal stress of 20kPa, the tests were conducted with a thickness of 3 mm for the soil specimen.

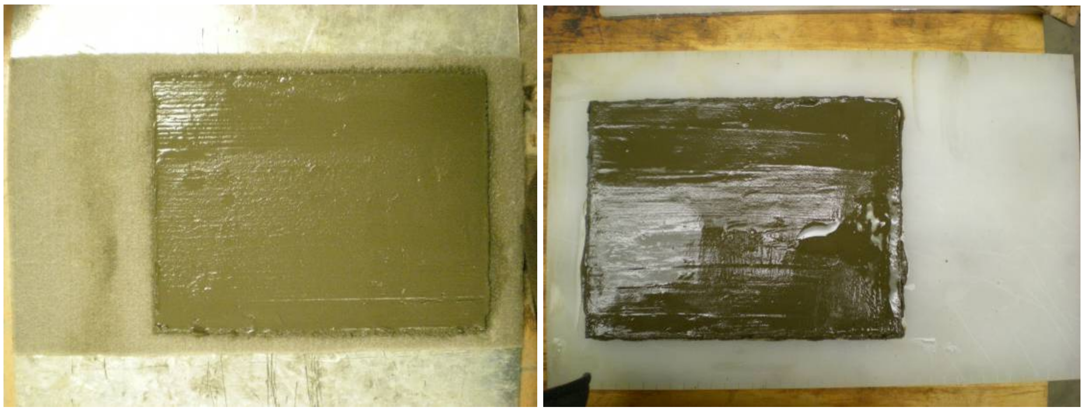


Figure 4.2 Specimen of Clay Spread on the Interface (Geotextile / Smooth interface)

4.3 Consolidation

After spreading the soils uniformly on the interface, the upper loading plate is placed on the top of the soil specimen (Figure 4.3). The upper loading plate is submerged for 30 minutes to ensure that the geotextile at the bottom of the loading plate is fully saturated. In order to eliminate the possibility of bearing capacity failure at the edge of the soil specimen, the soil is consolidated in two steps. First, the specimen is loaded with one-half of the desired normal stress and then left to consolidate for the amount of time needed to reach a degree of consolidation of 95 percent. Based on assumption of one-dimensional consolidation, the time required for a degree of consolidation of 95 percent is calculated by Eq. 4-1.

$$\mathbf{t_{95}} = \frac{\mathbf{T_{95} * d^2}}{\mathbf{c_\upsilon}} \qquad \textbf{Eq. 4-1}$$

where T_{95} = 1.129 (time factor with degree of consolidation, Terzaghi 1936), c_υ = coefficient of consolidation, d = drainage path length which is the thickness of soil specimen in this study (d = one-half of the thickness of soil specimen for the tests aimed at measuring the drained residual shear strength of soils because the geotextile is used at both sides of the soil specimens), t_{95} is the time to give a degree of consolidation of 95 percent. The coefficient of consolidation (c_υ) can be determined from laboratory consolidation data. The coefficient of consolidation is not a constant, but varies with both the level of stress and degree of consolidation. The value of c_υ obtained from 1-D consolidation tests is approximately 0.5 to 3 m^2/yr for the marine clays used in this study for normal stresses ranging from 0.25 to 20 kPa (Bae, Cheon et al. 2009). For the kaolinite, the coefficient of consolidation on remolded specimens varied between 0.5 and 2.0 m^2/yr for effective normal stresses used in this study (Sridharan and Nagaraj 2004). For the worst case (3 mm in thickness, 0.5 of c_υ, and interface tests), the time required to achieve a degree of consolidation of 95 percent is conservatively calculated from Eq. 4-1 as:

$$t_{95} = \frac{T_{95} * d^2}{c_\upsilon} = \frac{(1.129 * (0.003\text{m})^2)}{0.5\text{m}^2/\text{yr}} = 10.7 \text{ minutes}$$

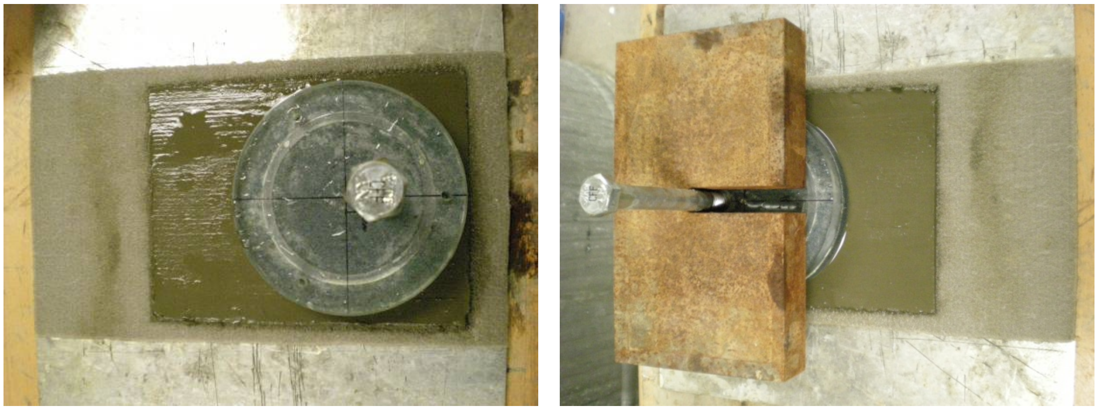


Figure 4.3 Consolidation Stage (For clarity, the picture was taken outside of the bath)

The soil specimen is inserted into the water bath in a galvanized steel tub a width of 6.1m, a length of 12.2m, and a height of 6.1m. The total desired normal stress is then applied and left to consolidate for over 30 minutes to ensure that the soil specimen is consolidated with the degree of consolidation of at least 95 percent. The consolidation time of 30 minutes is enough to achieve equilibrium under the effective normal stresses used in this study based on the above calculation.

Figure 4.4 Tilt Table Test Device in Water Bath

For undrained shear tests, an impermeable geomembrane is placed at the bottom of the loading plate and rough pipeline coating is used so that no drainage is allowed from the top and bottom of the soil specimen. Therefore, water can only travel in the horizontal plane, 152mm in diameter, to drain out of the sides of the specimen. The time required to achieve a degree of consolidation of 95 percent is conservatively calculated as a one-dimensional consolidation problem with radial drainage using following equation (Najjar, Gilbert et al. 2007).

$$\mathbf{t_{95}} = \frac{\mathbf{T_{95} * R^2}}{\mathbf{c_r}} = \frac{\mathbf{(0.375 * (0.076m)^2)}}{\mathbf{0.5m^2/yr}} = \mathbf{38\ hours} \qquad \textbf{Eq. 4-2}$$

where $T_{95} = 0.375$ (assuming equal strain), C_r = radial coefficient of consolidation, R = radius of upper loading plate (Gibson and Henkel 1954).

4.4 Shearing

After the consolidation stage is complete, the shear stress is applied by lifting the base plate from the horizontal. During this process for drained conditions, the tilt table must be lifted at a slow enough rate to insure complete dissipation of excess pore water pressures induced by shear stress. The following standard for the total elapsed time to failure, t_f, required for achieving drained conditions in a direct shear test provides guidance in determining rate of loading (ASTM D3080 2004):

$$\mathbf{t_f = 50 * t_{50}} \qquad \textbf{Eq. 4-3}$$

where t_{50} = time required for achieving 50 percent consolidation is calculated by Eq. 4-4.

$$\mathbf{t_{50} = \frac{T_{50} * d^2}{c_v}} \qquad \textbf{Eq. 4-4}$$

where T_{50} = 0.197 (time factor with degree of consolidation, Terzaghi 1936). For the c_v value of 0.5 m^2/yr, the soil thickness of 2mm, and interface tests, t_{50} is about 0.83 minutes and the time to failure t_f is approximately 42 minutes. This time to failure is the minimum interval needed to achieve drained shear status based on a constant rate of deformation. The tilt table device corresponds to a load-controlled test. Since the rate of deformation in the soil is expected to increase as the base plate becomes more inclined, the load increments are decreased to maintain approximately a constant level of deformation as the soil specimen approaches failure. Special care and time are needed to measure the friction angle at the first failure because this first estimate of the friction angle is used to establish the loading rate allowing the soil specimen to drain and consolidate under each load increment for a specified period of time. The cumulative sum of the time intervals must be larger than the actual time to failure calculated by Eq. 4-3 to insure equilibrium under the applied shear stresses. The total number of load increments depends on the time to failure given by Eq. 4-3. The excess pore water pressure induced by the shear force at low levels of angles is less than that at the high angles. From this point of view, a minimum load increment of about 0.5° in tilt can be obtained as a practical lower bound, whereas an upper bound of the load increment is about 5° (Najjar, Gilbert et al. 2007).

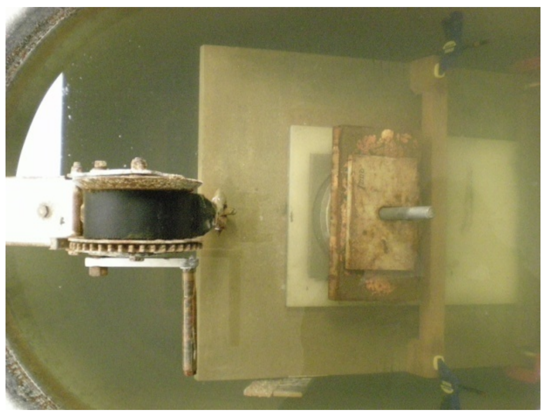


Figure 4.5 Shearing Stage

For undrained shear tests, the time required to limit the dissipation of excess pore water pressures during the shearing process to less than 5 percent can be calculated using following equation (Gibson and Henkel 1954).

$$\mathbf{t = \frac{R^2}{8 * c_r * (1-U)} = \frac{(0.076m)^2}{8 * \frac{0.5m^2}{yr} * (1-0.05)} = 13\ hours} \qquad \textbf{Eq. 4-5}$$

where U = 0.05. When the time to failure is less than t (13 hours) calculated above, the total dissipation of excess pore water pressures during shearing process will be less than 5 percent. In order to minimize the dissipation of pore water pressures induced by shearing, the shearing process is conducted with a quick enough rate so that this time to failure is achieved.

4.5 Practical Test Procedure

The following procedure is adopted for the shearing step to maintain drained conditions in the soils. For the first four increments, the tilt table angle is increased in 5 degree steps and left to drain for 5 minutes per increment. After then, the load step is cut down to 2 degrees for another five increments and left to drain for 5 minutes per increment. A careful attention is paid to this process especially for the first failure. Trial and error was used with different interval of increment for these tests and the loading step was cut down to 1 degree until failure occurred. After the initiation of failure, the loading plate is allowed to slide for 15mm using a wood stopper. The tilt table is then lowered by 10 degrees and the whole procedure repeated until the residual condition is reached.

For the undrained tests, the time needed to achieve a degree of consolidation of 95 percent is 38 hours. Since this time interval seemed to not practical for this study, two steps are adopted for the consolidation stage as follows: (1) before submerging the soil specimen into the water bath, the geotextile is introduced at the bottom of the loading plate to fully consolidate the soil for 30 minutes; (2) the applied surcharge weight and upper loading plate are removed, and then the impermeable geomembrane is placed on the top of the soil specimen. After inserting the soil specimen into the water bath, the specimen is reconsolidated for another 30 minutes before initiating the shearing stage. Careful attention is paid to replace the loading plate to ensure the original placement. The time needed to limit the dissipation of excess pore water pressures during shearing process to less than 5 percent is 13 hours, based on the assumption that the drainage can only occur in the horizontal direction. The shearing process should be conducted at a quick enough rate to minimize a dissipation of excess pore water pressures induced by shearing stress. However, in order to eliminate time-rate effects on the measured undrained shear strength, the time to failure of 30 minutes is used in these tests.

For the normal stress less than 2 kPa, the residual friction angle could not be measured precisely where the angle of friction is larger than that of the tilting table tests. For example, the secant friction angle of the first failure for the soil from source BC3 at the effective normal stress of 0.25 kPa is 57°. In this case, the base plate is detached from the gear and controlled by hand to determine the angle.

The test is continued until more than 75mm of the total displacement are attained to ensure residual conditions. Based on the above procedure, the minimum time to failure obtained in this study for drained shear tests is about 60 minutes, which is in excess of 42 minutes, estimated time to failure by Eq. 4-3.

4.6 Deformation Control

Two wood stoppers are used to adjust the amount of deformations when the upper loading plate slides down under applied shear stresses (Figure 4.6). Each wood stopper has the same width with the base plate (460 mm) and a height of 30 and 60 mm respectively. A wood stopper is fixed with two clamps at the desired position on the base plate so that the wood stopper stops the upper loading plate after about 15mm deformations for each failure. When the soil reaches the residual condition, more than two successive failures will occur at the same angle. In general, the residual strength is obtained at a total displacement of about 50 mm. Therefore, in order to ensure that the residual strength is reached, all tests are continued until more than 75 mm (five times of failure) of deformation is attained. Note that the maximum travel distance of the upper loading plate is about 100 mm given the geometry of soil specimen.

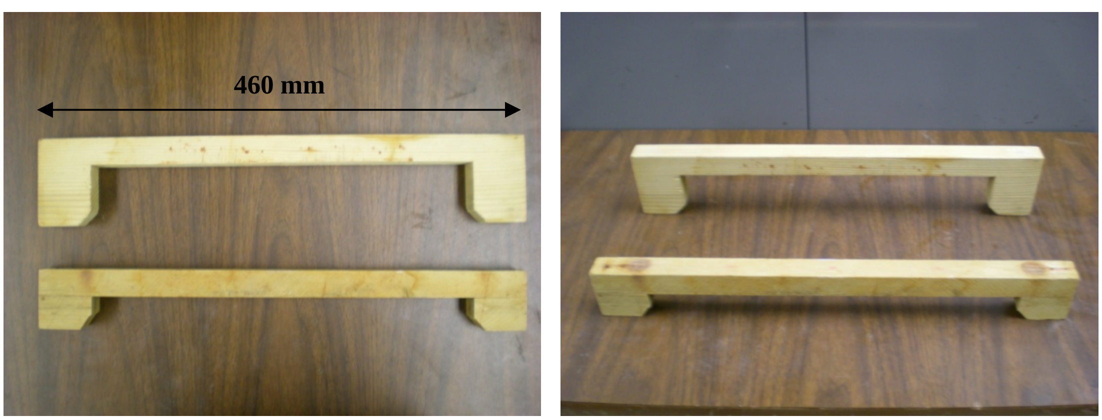


Figure 4.6 Wood Stopper

4.7 Loading Eccentricity

Since the applied forces are limited by the surcharge weight and the angle of inclination, the normal stresses acting on the soil specimen decrease as the base plate is tilted, whereas the shear stresses driving the failure increase. The effective normal stress and the shear stress are calculated by following equations respectively.

Effective Normal Stress: $\sigma' = (W'/A) \cos\beta$

Shear Stress: $\tau = (W'/A) \sin\beta$

where: W' is the submerged weight of the upper loading plate acting on the soil

A is the area of the loading plate

β is the angle of inclination of the tilt table at failure

One possible limitation of the tilt table test is that nonuniform normal stresses develop along the interface due to the overturning moment induced by the eccentric loading as the table is inclined. Moreover, the magnitude of the eccentricity increases with increasing the angle of inclination and applied surcharge weight (the height of weight). Without adjusting for eccentricity, the upper loading plate will not remain parallel to the base plate at failure and the top of the loading plate could be lifted up by the overturning moment.

In order to minimize the effect of eccentricity: (1) the height of weight placed on the upper loading plate is minimized so that the center of gravity is located as close as possible to the soil specimens; (2) the center of gravity for the applied load is shifted back by 18 mm so that the net eccentricity at failure would be close to zero (Figure 4.7). The eccentricity is estimated with an expected friction angle for each test. With trial and error, the final position of the center of gravity is decided by estimating the worst case that corresponds to the highest friction angle at the highest normal stress used in this study. In addition, visual inspection is conducted during the test to ensure that there is no sign of noticeable pitch of the upper loading plate that should remain parallel to the base plate.

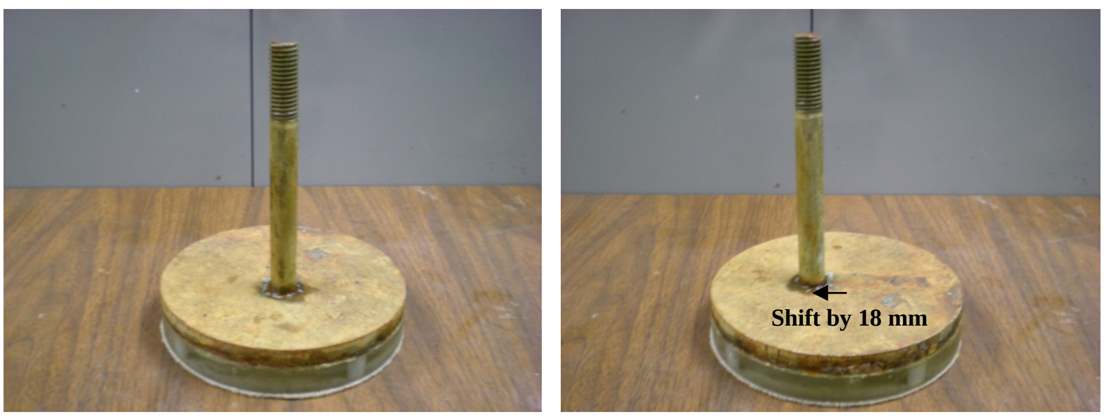


Figure 4.7 Shifted Position of Center of Gravity

Chapter 5 Test Program

The objectives of the tilt table tests conducted in this study are: (1) to measure the drained residual shear and interface strength of soils at low effective normal stresses ranging from 0.25 to 20 kPa; (2) to investigate the effect of several parameters, such as effective normal stress, stress history, loading rate, roughness of interface and composition of soils on drained residual shear strength; (3) to study the correlations between the drained residual shear strength and index properties, clay content, and clay mineralogy. Two series of tests have been performed in this study.

For the first series of tests, the marine clays from three different locations at an offshore project site are used. The three soils are referred to in this study as BC1, BC2, and BC3. In the second series of tests, Monterey #30 sand and Kaolinite are used to simulate the properties of normal clays. The effective normal stresses used in these tests are 2, 6, and 20kPa. The Kaolinite is thoroughly mixed with sand in the dry powdered state in different proportions ranging from 10 to 70 percent by weight. The soil mixtures are referred to in this study as KM_10 to KM_70. The tilt table tests are performed with soil mixtures and two materials (pure kaolinte and pure sand) are tested separately for comparison purposes. Additional six tests are performed to evaluate the effect of pore-water chemistry on the drained residual shear strength of marine clays.

5.1 Test Materials

5.1.1 Soil

Index properties for all soil sources are presented in Table 5.1. Marine clays from three different locations at a project site were provided in the large plastic bags. Clay from source BC1 and BC2 classify as extremely high plasticity silt (ME) based on Unified Soil Classification System (Bae, Cheon et al. 2009). The water contents measured in the laboratory after thorough mixing of the soil from each bag were approximately 100 percent and 140 percent respectively. Clay from source BC3 classifies as extremely high plasticity clay (CE) and the as-delivered water content was about 160 percent. Visual inspection of the clay samples reveals that the clay samples have some shell fragments that were removed before conducting the tests.

In order to examine the soil behavior with a wide range of clay contents and a sand-clay rather than a sand-silt-clay mixture, tests are performed with sand, kaolinite, and sand-kaolinite mixtures. The kaolinite and Monterey #30 sand are used in these tests. The Kaolinite is prepared to a target water content of 70 percent of the dry weight by adding tap water and mixing thoroughly to achieve homogeneity. The sand is fully saturated with tap water and placed on the interface as mentioned in Chapter 4.2. The sands classify as poorly graded sand (SP) based on the results of sieve analysis (Figure 5.1). All the mixtures are first mixed in a dry state and saturated with a designated amount of tap water (70 percent of the dry weight of kaolinite). Index properties for all mixtures are also presented in Table 5.1. All soil samples are stored in sealable plastic bags to maintain initial water contents as close as possible.

Table 5.1 Index Properties for Soils

Soil Source	Soil Classification	LL(%)	PL(%)	PI(%)	Clay(%)	Silt(%)	Sand(%)	Initial w(%)
BC1	ME	102	51	51	9	84	7	90~102
BC2	ME	135-144	56-59	76-88	9	79	12	138~167
BC3	CE	132-141	45-48	87-93	16	56	28	162~164
Kaolinite	CH	56	31	25	100	0	0	70
KM_10	ML	6	3	2	10	0	90	10
KM_30	CL	17	9	7	30	0	70	20
KM_50	CL	28	16	12	50	0	50	35
KM_60	CL	34	19	15	60	0	40	42
KM_70	CI	39	22	17	70	0	30	50
Monterey #30	SP	N/A			0	0	100	27

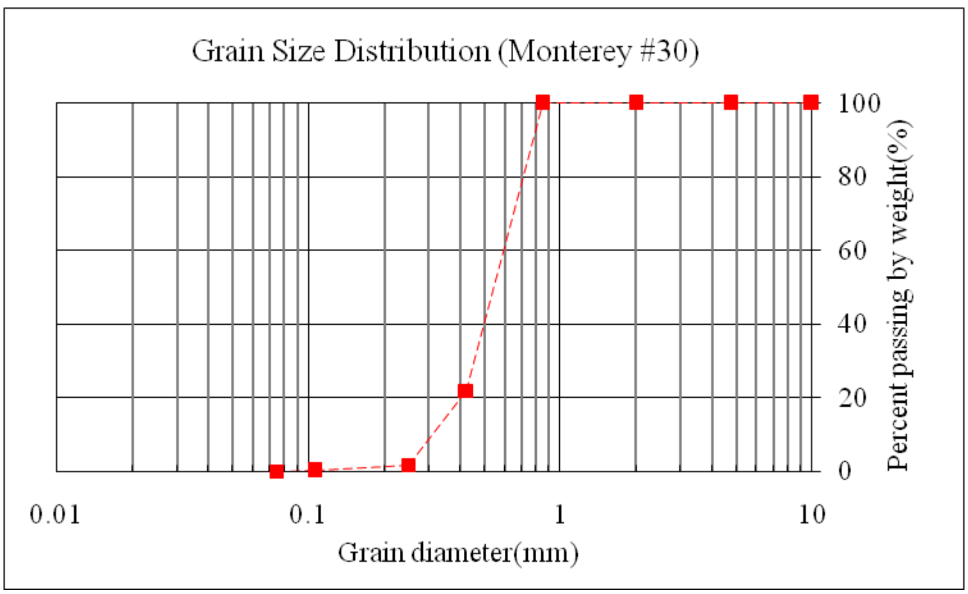


Figure 5.1 Size Distribution of Monterey #30 Sand

5.1.2 *Pore Water*

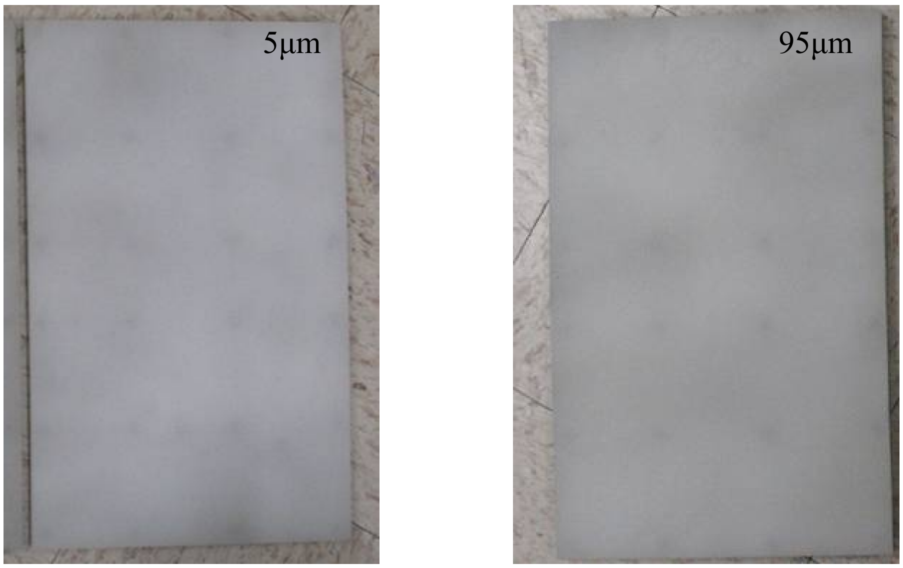
For the marine clay samples (BC1, BC2, and BC3), the specimens are mixed and tested in salt water that was prepared with a sea salt salinity of 35 parts per thousand (35 g/l), simulating pore-water that already was present. For the kaolinite, sand, and sand-kaolinite mixtures, the specimens are mixed and tested in tap water. The measured salinities of each pore-water are presented in Figure 5.2.

(a) Tap Water (12 g/l) (b) Salt Water (36 g/l)

Figure 5.2 Salinity in Pore Fluid (Tap / Salt Water)

5.1.3 *Interfaces*

For the tests aimed at measuring the drained residual shear strength at interface between soils and pipeline coatings, two types of pipeline coatings are used in this study. Both interfaces are white coatings bonded to a flat steel plate that are 25 x 40 cm in area and about 0.8 cm in thickness. Profile measurement indicates that the CLA values of these interfaces are approximately 5 and 95 μm, respectively. The interfaces are referred to in this study as smooth (5μm) and rough (95μm). For the undrained shear tests, the rough coating and impermeable geomembrane are used to prevent the drainage from the top and bottom of the specimen during shearing (Figure 5.3).



(a) Smooth (b) Rough

Figure 5.3 Pipeline Interfaces (Smooth / Rough)

For the tests aimed at measuring the drained residual shear strength of soil against soil, the geotextile epoxy-glued on an aluminum plate is used as an interface material as shown in Figure 5.4. This interface provides freely draining conditions at the bottom of the soil specimen.

Figure 5.4 Geotextile Interface

5.2 Tests on Marine Clays

A total of 28 tilt table tests are carried out with marine clays (Table 5.2). Eighteen tests are performed to measure drained residual shear strength of clays. In these tests, a geotextile is attached to the base plate instead of the pipeline coating. These tests indicate that the failure surface will be created in the middle of clay and the measured residual strength represents the internal strength of clay. Ten tests are conducted on the rough and smooth pipeline coatings to measure the drained residual shear strength at the interface.

Table 5.2 Summary of Test Program on Marine Clays

NO.	Test ID	Soil Source	Soil Classification	Pore water	Test type	Thickness (mm)	Interface	N stress at horizontal(kPa)	Surcharge Weight	Initial w(%)
1	BC1_0	BC1	ME	Salt water	CD	2	Geotextile	0.25	D6A+250	90
2	BC1_2	BC1	ME	Salt water	CD	2	Geotextile	2.01	D6S+2000	91
3	BC1_4	BC1	ME	Salt water	CD	2	Geotextile	4.01	D6S+5000	102
4	BC1_6	BC1	ME	Salt water	CD	2	Geotextile	5.98	D6S+8000	100
5	BC1_10	BC1	ME	Salt water	CD	2	Geotextile	11.25	D6S+16000	90
6	BC1_20	BC1	ME	Salt water	CD	3	Geotextile	21.79	D6S+32000	90
7	BC2_0	BC2	ME	Salt water	CD	2	Geotextile	0.25	D6A+250	142
8	BC2_2	BC2	ME	Salt water	CD	2	Geotextile	2.01	D6S+2000	138
9	BC2_4	BC2	ME	Salt water	CD	2	Geotextile	4.01	D6S+5000	168
10	BC2_6	BC2	ME	Salt water	CD	2	Geotextile	5.98	D6S+8000	167
11	BC2_10	BC2	ME	Salt water	CD	2	Geotextile	11.25	D6S+16000	167
12	BC2_20	BC2	ME	Salt water	CD	3	Geotextile	21.79	D6S+32000	167
13	BC3_0	BC3	CE	Salt water	CD	2	Geotextile	0.25	D6A+250	162
14	BC3_2	BC3	CE	Salt water	CD	2	Geotextile	2.01	D6S+2000	164
15	BC3_4	BC3	CE	Salt water	CD	2	Geotextile	4.01	D6S+5000	164
16	BC3_6	BC3	CE	Salt water	CD	2	Geotextile	5.98	D6S+8000	164
17	BC3_10	BC3	CE	Salt water	CD	2	Geotextile	11.25	D6S+16000	164
18	BC3_20	BC3	CE	Salt water	CD	3	Geotextile	21.79	D6S+32000	162
19	BC1_2	BC1	ME	Salt water	CD	2	Rough	2.01	D6S+2000	91
20	BC1_4	BC1	ME	Salt water	CD	2	Rough	4.01	D6S+5000	102
21	BC1_6	BC1	ME	Salt water	CD	2	Rough	5.98	D6S+8000	100
22	BC2_2	BC2	ME	Salt water	CD	2	Rough	2.01	D6S+2000	138
23	BC2_4	BC2	ME	Salt water	CD	2	Rough	4.01	D6S+5000	168
24	BC2_6	BC2	ME	Salt water	CD	2	Rough	5.98	D6S+8000	167
25	BC3_2	BC3	CE	Salt water	CD	2	Rough	2.01	D6S+2000	164
26	BC3_4	BC3	CE	Salt water	CD	2	Rough	4.01	D6S+5000	164
27	BC2_4	BC2	ME	Salt water	CD	2	Smooth	4.01	D6S+5000	168
28	BC2_6	BC2	ME	Salt water	CD	2	Smooth	5.98	D6S+8000	167

CD = Consolidated-Drained Shear Test

5.3 Tests on Sand, Kaolinite, and Sand-Kaolinite Mixtures

The purpose of the tests on sand, kaolinite and sand-kaolinite mixtures is similar to that of the tests on the marine clays. In addition, the tests are intended to examine the drained residual shear strength of soils with a wide range of clay contents, and with a sand-clay rather than a sand-silt-clay mixture. A total of 46 tilt table tests are performed with sand, kaolinite, and sand-kaolinite mixtures (Table 5.3). Fifteen tests are performed using the soil mixtures to examine the effect the clay contents on drained residual shear strength of soils. Twelve tests are conducted using smooth pipeline coating instead of geotextile layer to evaluate the effect of clay contents on the coating efficiency under different effective normal stress levels. Another twelve tests are carried out to measure the drained residual strength of pure clay and sand for comparison purposes. In addition, six tests are performed using Ottawa sand to examine the effect of size of sand particles on the drained residual shear strength of cohesionless soils (Appendix B).

For the tilt table test method, the normal stresses at failure are not equal to the nominal values since the applied normal stress decreases with inclination (θ), meaning that all soil specimens are overconsolidated at failure and the overconsolidation ratio (OCR) becomes $1/\cos\theta$ (Pedersen, Olson et al. 2003). All tests are conducted by consolidating the soil after submerging the apparatus and the applied surcharge weight to minimize the OCR effect. The OCR at failure in these tests ranges from 1.1 to 1.4. In order to investigate the effect of overconsolidation on drained residual shear strength of clays, three additional tests are carried out using Kaolinite with different values of over-consolidation ratio (OCR).

In order to assure that the tile table test method is measuring the drained residual shear strength of soils, undrained shear tests together with two values of OCR and a creep test are performed. For undrained shear tests, since the rough pipeline coating is used as an interface material instead of the geotextile layer, two drained interface tests are performed to evaluate the measured residual strength under rapid loading with the same pipeline coating.

Table 5.3 Summary of Test Program on Sand, Kaolinite, and Sand-Kaolinite Mixtures

NO.	Test ID	Kaolinite(%)	Monterey#30 Sand(%)	Soil Classification	Pore water	Test type	Thickness (mm)	Interface	N stress at horizontal(kPa)	Surcharge weight
1	KM_10_2	10	90	ML	Tap water	CD	2	Geotextile	2.01	D6S+2000
2	KM_30_2	30	70	CL	Tap water	CD	2	Geotextile	2.01	D6S+2000
3	KM_50_2	50	50	CL	Tap water	CD	2	Geotextile	2.01	D6S+2000
4	KM_60_2	60	40	CL	Tap water	CD	2	Geotextile	2.01	D6S+2000
5	KM_70_2	70	30	CI	Tap water	CD	2	Geotextile	2.01	D6S+2000
6	KM_10_6	10	90	ML	Tap water	CD	2	Geotextile	5.98	D6S+8000
7	KM_30_6	30	70	CL	Tap water	CD	2	Geotextile	5.98	D6S+8000
8	KM_50_6	50	50	CL	Tap water	CD	2	Geotextile	5.98	D6S+8000
9	KM_60_6	60	40	CL	Tap water	CD	2	Geotextile	5.98	D6S+8000
10	KM_70_6	70	30	CI	Tap water	CD	2	Geotextile	5.98	D6S+8000
11	KM_10_20	10	90	ML	Tap water	CD	3	Geotextile	21.79	D6S+32000
12	KM_30_20	30	70	CL	Tap water	CD	3	Geotextile	21.79	D6S+32000
13	KM_50_20	50	50	CL	Tap water	CD	3	Geotextile	21.79	D6S+32000
14	KM_60_20	60	40	CL	Tap water	CD	3	Geotextile	21.79	D6S+32000
15	KM_70_20	70	30	CI	Tap water	CD	3	Geotextile	21.79	D6S+32000
16	KM_10_2	10	90	ML	Tap water	CD	2	Smooth	2.01	D6S+2000
17	KM_70_2	70	30	CI	Tap water	CD	2	Smooth	2.01	D6S+2000
18	KM_10_6	10	90	ML	Tap water	CD	2	Smooth	5.98	D6S+8000
19	KM_70_6	70	30	CI	Tap water	CD	2	Smooth	5.98	D6S+8000
20	KM_10_20	10	90	ML	Tap water	CD	3	Smooth	21.79	D6S+32000
21	KM_70_20	70	30	CI	Tap water	CD	3	Smooth	21.79	D6S+32000
22	M_2	0	100	SP	Tap water	CD	2	Smooth	2.01	D6S+2000
23	M_6	0	100	SP	Tap water	CD	2	Smooth	5.98	D6S+8000
24	M_20	0	100	SP	Tap water	CD	3	Smooth	21.79	D6S+32000
25	K_2	100	0	CH	Tap water	CD	2	Smooth	2.01	D6S+2000
26	K_6	100	0	CH	Tap water	CD	2	Smooth	5.98	D6S+8000
27	K_20	100	0	CH	Tap water	CD	3	Smooth	21.79	D6S+32000
28	K_0	100	0	CH	Tap water	CD	2	Geotextile	0.25	D6A+250
29	K_2	100	0	CH	Tap water	CD	2	Geotextile	2.01	D6S+2000
30	K_4	100	0	CH	Tap water	CD	2	Geotextile	4.01	D6S+5000
31	K_6	100	0	CH	Tap water	CD	2	Geotextile	5.98	D6S+8000
32	K_10	100	0	CH	Tap water	CD	2	Geotextile	11.25	D6S+16000
33	K_20	100	0	CH	Tap water	CD	3	Geotextile	21.79	D6S+32000
34	M_0	0	100	SP	Tap water	CD	2	Geotextile	0.25	D6A+250
35	M_2	0	100	SP	Tap water	CD	2	Geotextile	2.01	D6S+2000
36	M_4	0	100	SP	Tap water	CD	2	Geotextile	4.01	D6S+5000
37	M_6	0	100	SP	Tap water	CD	2	Geotextile	5.98	D6S+8000
38	M_10	0	100	SP	Tap water	CD	2	Geotextile	11.25	D6S+16000
39	M_20	0	100	SP	Tap water	CD	3	Geotextile	21.79	D6S+32000
40	K_6	100	0	CH	Tap water	CD	2	Geotextile	2.01	8000->2000
41	K_10	100	0	CH	Tap water	CD	2	Geotextile	2.01	16000->2000
42	K_20	100	0	CH	Tap water	CD	2	Geotextile	2.01	32000->2000
43	K_2	100	0	CH	Tap water	CD	2	Rough	2.01	D6S+2000
44	K_20	100	0	CH	Tap water	CD	2	Rough	21.79	D6S+32000
45	KU_2	100	0	CH	Tap water	CU	2	Rough	2.01	D6S+2000
46	KU_20	100	0	CH	Tap water	CU	2	Rough	2.01	D6S+2000

CU = Consolidated-Undrained Shear Test

Chapter 6 Test Results and Data Analysis

6.1 Introduction

This chapter presents the test results from this study. The individual test results are provided in Table 6.1 and 6.2, and shown as a function of effective normal stress in Figure 6.1, 6.2, and 6.3. All test results are expressed as the measured value of drained residual secant friction angles of the soils. The secant friction angle is defined as the arc tangent of the ratio of the shear stress to the effective normal stress at failure. These angles are converted to the friction coefficients defined as the ratio of the shear stress to the effective normal stress at failure. The test results also include the effect of various parameters on the drained residual shear and interface strength of soils. The interface results are evaluated in terms of the efficiency of the coating. The coating efficiency is defined as the ratio of the shear strength of interface to shear strength of soil (Eq. 6-1).

$$\textbf{coating efficiency} = \frac{\textbf{shear strength of interface}^{1}}{\textbf{shear strength of soil}^{2}} \qquad \textbf{Eq. 6-1}$$

6.2 Test Results

6.2.1 Test Results of Marine Clays

Results contain the drained residual shear and interface strength for soils from source BC1, BC2, and BC3. Figure 6.1 exhibits the typical failure envelope and friction coefficients for soils under the effective normal stress ranging from 0.25 to 20 kPa.

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[1] Drained residual shear strength of soils tested on a smooth pipeline coating.
[2] Drained residual shear strength of soils tested on a nonwoven, needle-punched geotextile.

Table 6.1 Summary of Test Results of Marine Clays

NO.	Test ID	Soil Source	N stress at horizontal(kPa)	Failure type	N stress at failure (kPa)	Residual secant friction angle	S stress at failure (kPa)	OCR	Coefficient of friction
1	BC1_0	BC1	0.25	Internal	0.2	43.5	0.2	1.4	0.9
2	BC1_2	BC1	2.01	Internal	1.6	38	1.2	1.3	0.8
3	BC1_4	BC1	4.01	Internal	3.2	36	2.4	1.2	0.7
4	BC1_6	BC1	5.98	Internal	4.9	34.5	3.4	1.2	0.7
5	BC1_10	BC1	11.25	Internal	9.4	33.5	6.2	1.2	0.7
6	BC1_20	BC1	21.79	Internal	18.5	32	11.5	1.2	0.6
7	BC2_0	BC2	0.25	Internal	0.2	43.5	0.2	1.4	0.9
8	BC2_2	BC2	2.01	Internal	1.6	38	1.2	1.3	0.8
9	BC2_4	BC2	4.01	Internal	3.2	37	2.4	1.3	0.8
10	BC2_6	BC2	5.98	Internal	4.8	36	3.5	1.2	0.7
11	BC2_10	BC2	11.25	Internal	9.3	34.5	6.4	1.2	0.7
12	BC2_20	BC2	21.79	Internal	18.3	33	11.9	1.2	0.6
13	BC3_0	BC3	0.25	Internal	0.2	45	0.2	1.4	1.0
14	BC3_2	BC3	2.01	Internal	1.5	40	1.3	1.3	0.8
15	BC3_4	BC3	4.01	Internal	3.1	39	2.5	1.3	0.8
16	BC3_6	BC3	5.98	Internal	4.8	37	3.6	1.3	0.8
17	BC3_10	BC3	11.25	Internal	9.1	36	6.6	1.2	0.7
18	BC3_20	BC3	21.79	Internal	18.0	34.5	12.3	1.2	0.7
19	BC1_2	BC1	2.01	Internal	1.6	38	1.2	1.3	0.8
20	BC1_4	BC1	4.01	Internal	3.2	36	2.4	1.2	0.7
21	BC1_6	BC1	5.98	Internal	4.9	34.5	3.4	1.2	0.7
22	BC2_2	BC2	2.01	Internal	1.6	38	1.2	1.3	0.8
23	BC2_4	BC2	4.01	Internal	3.2	37	2.4	1.3	0.8
24	BC2_6	BC2	5.98	Internal	4.8	36	3.5	1.2	0.7
25	BC3_4	BC3	4.01	Internal	3.1	39	2.5	1.3	0.8
26	BC3_6	BC3	5.98	Internal	4.8	37	3.6	1.3	0.8
27	BC2_4	BC2	4.01	Interface	3.2	36	2.4	1.2	0.7
28	BC2_6	BC2	5.98	Interface	4.9	35	3.4	1.2	0.7

OCR = Ratio of normal stress for consolidation to normal stress during shear

N stress = Effective Normal Stress, S stress = Residual Shear Stress

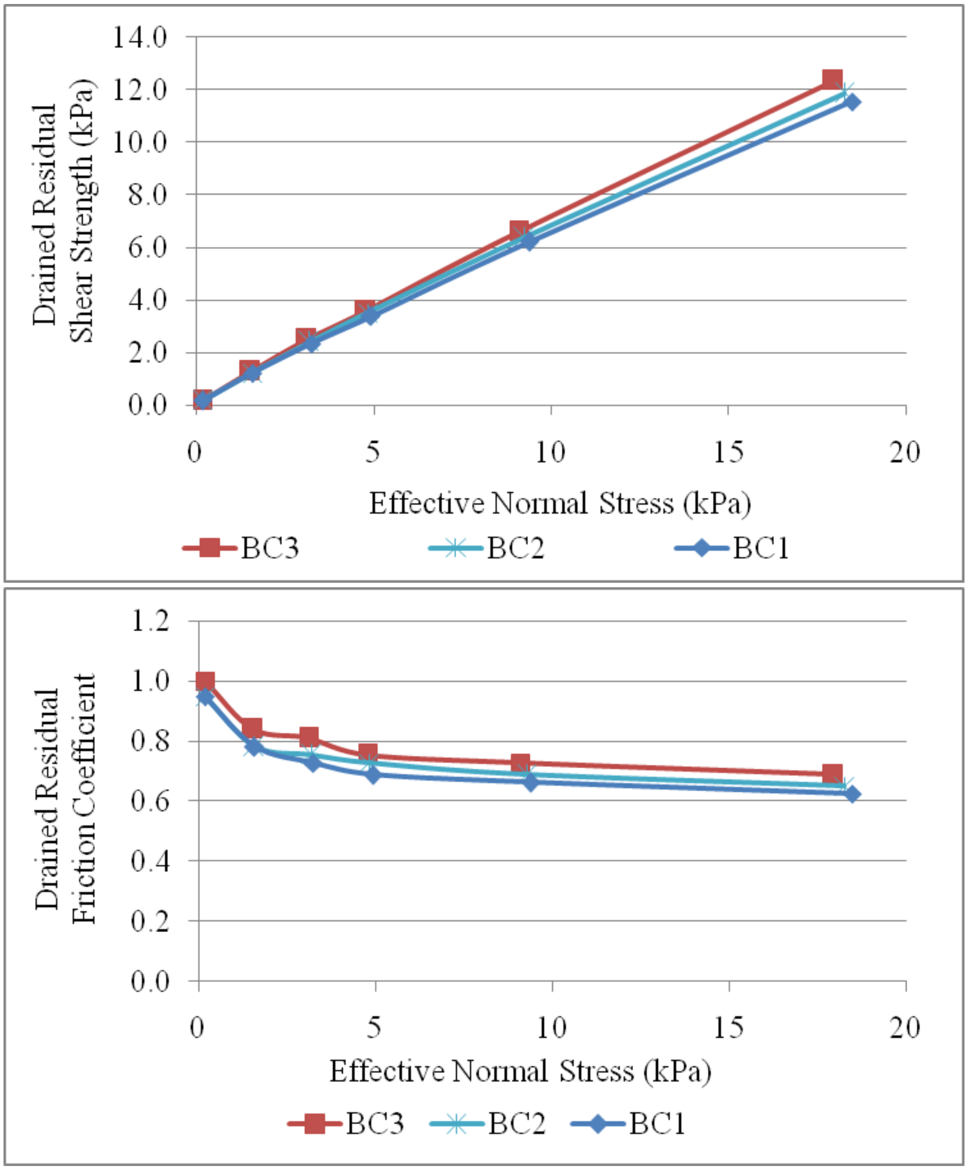


Figure 6.1 Summary of Test Results of Marine Clays

6.2.2 Test Results of Sand, Kaolinite, and Sand-Kaolinite Mixtures

Results in the second series of tests mainly consist of two parts: (1) the drained residual shear strength of sand, kaolinite, and sand-kaolinite mixtures; (2) the coating efficiency against smooth interface with low and high clay fraction. These test results are summarized in Table 6.2. Figure 6.2 and 6.3 exhibit the typical failure envelope and friction coefficients for sand, kaolinite and sand-kaolinite mixtures under the effective normal stress of 2, 6, and 20 kPa. The failure envelope of Kaolinite intersects that of Monterey #30 sand at the effective normal stress of 6 kPa.

Table 6.2 Summary of Test Results on Sand, Kaolinite, and Sand-Kaolinite Mixtures

NO.	Test ID	N stress at horizontal(kPa)	Failure type	N stress at failure (kPa)	Residual secant friction angle	S stress at failure (kPa)	OCR	Coefficient of friction	w(%) at failure surface
1	KM_10_2	2.01	Internal	1.8	24	0.8	1.1	0.45	17
2	KM_30_2	2.01	Internal	1.8	28	0.9	1.1	0.53	20
3	KM_50_2	2.01	Internal	1.8	29	1.0	1.1	0.55	27
4	KM_60_2	2.01	Internal	1.7	31.5	1.1	1.2	0.61	28
5	KM_70_2	2.01	Internal	1.7	33.6	1.1	1.2	0.66	35
6	KM_10_6	5.98	Internal	5.5	24	2.4	1.1	0.45	18
7	KM_30_6	5.98	Internal	5.4	24.5	2.5	1.1	0.46	18
8	KM_50_6	5.98	Internal	5.4	26	2.6	1.1	0.49	23
9	KM_60_6	5.98	Internal	5.3	27	2.7	1.1	0.51	27
10	KM_70_6	5.98	Internal	5.3	28	2.8	1.1	0.53	35
11	KM_10_20	21.79	Internal	19.8	24.5	9.0	1.1	0.46	17
12	KM_30_20	21.79	Internal	19.9	24	8.9	1.1	0.45	16
13	KM_50_20	21.79	Internal	20.1	23	8.5	1.1	0.42	20
14	KM_60_20	21.79	Internal	20.1	23	8.5	1.1	0.42	25
15	KM_70_20	21.79	Internal	20.2	22	8.2	1.1	0.40	32
16	KM_10_2	2.01	Internal	1.9	15	0.5	1.0	0.27	18
17	KM_70_2	2.01	Internal	1.7	31.5	1.1	1.2	0.61	40
18	KM_10_6	5.98	Combination	5.8	15	1.5	1.0	0.27	18
19	KM_70_6	5.98	Internal	5.5	24	2.4	1.1	0.45	27
20	KM_10_20	21.79	Combination	21.0	15	5.6	1.0	0.27	18
21	KM_70_20	21.79	Internal	20.5	20	7.5	1.1	0.36	49
22	M_2	2.01	Internal	1.8	24	0.8	1.1	0.45	27
23	M_6	5.98	Internal	5.5	24	2.4	1.1	0.45	27
24	M_20	21.79	Internal	20.1	23	8.5	1.1	0.42	27
25	K_2	2.01	Internal	1.7	31.5	1.1	1.2	0.61	62
26	K_6	5.98	Internal	5.4	25	2.5	1.1	0.47	37
27	K_20	21.79	Internal	20.5	20	7.5	1.1	0.36	50
28	K_0	0.25	Internal	0.2	39	0.2	1.3	0.8	62
29	K_2	2.01	Internal	1.6	36	1.2	1.2	0.7	60
30	K_4	4.01	Internal	3.3	33.6	2.2	1.2	0.7	60
31	K_6	5.98	Internal	5.2	30	3.0	1.2	0.6	58
32	K_10	11.25	Internal	9.9	28	5.3	1.1	0.5	55
33	K_20	21.79	Internal	19.9	24	8.9	1.1	0.4	49
34	M_0	0.25	Internal	0.2	30	0.1	1.155	0.58	27
35	M_2	2.01	Internal	1.8	29	1.0	1.1	0.55	28
36	M_4	4.01	Internal	3.5	28	1.9	1.133	0.53	27
37	M_6	5.98	Internal	5.2	29	2.9	1.1	0.55	27
38	M_10	11.25	Internal	9.8	29	5.5	1.143	0.55	27
39	M_20	21.79	Internal	19.1	29	10.6	1.1	0.55	27
40	K_6	2.01	Internal	1.6	36	1.2	**3.7**	0.7	
41	K_10	2.01	Internal	1.6	36	1.2	**6.9**	0.7	
42	K_20	2.01	Internal	1.6	37	1.2	**13.6**	0.8	
43	K_2	2.01	Internal	1.6	36	1.2	1.2	0.7	
44	K_20	21.79	Internal	19.9	24	8.9	1.1	0.4	
45	KU_2	2.01	Internal	1.9	20	0.7	1.1	0.4	
46	KU_20	2.01	Internal	1.6	36	1.2	**13.4**	0.7	

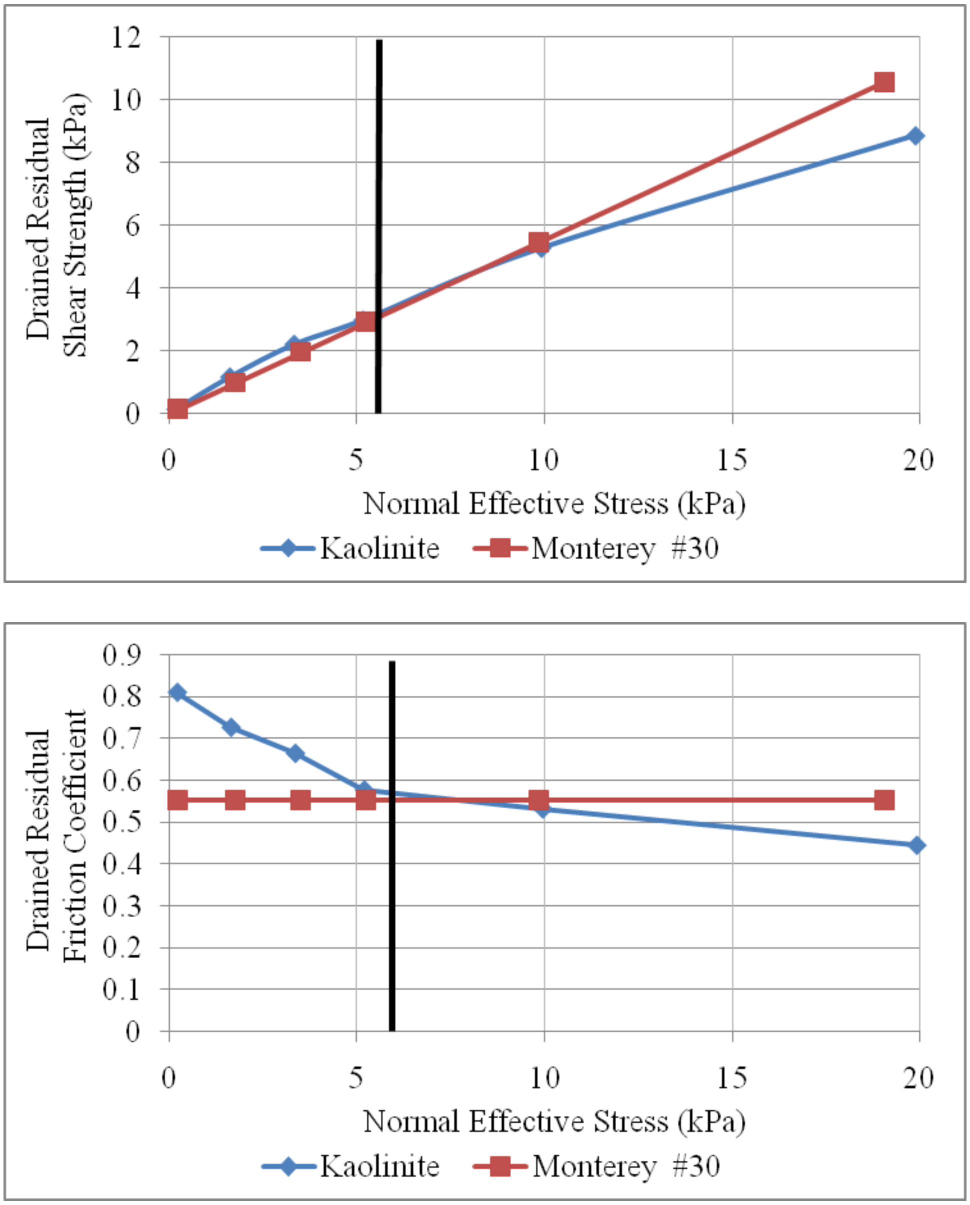
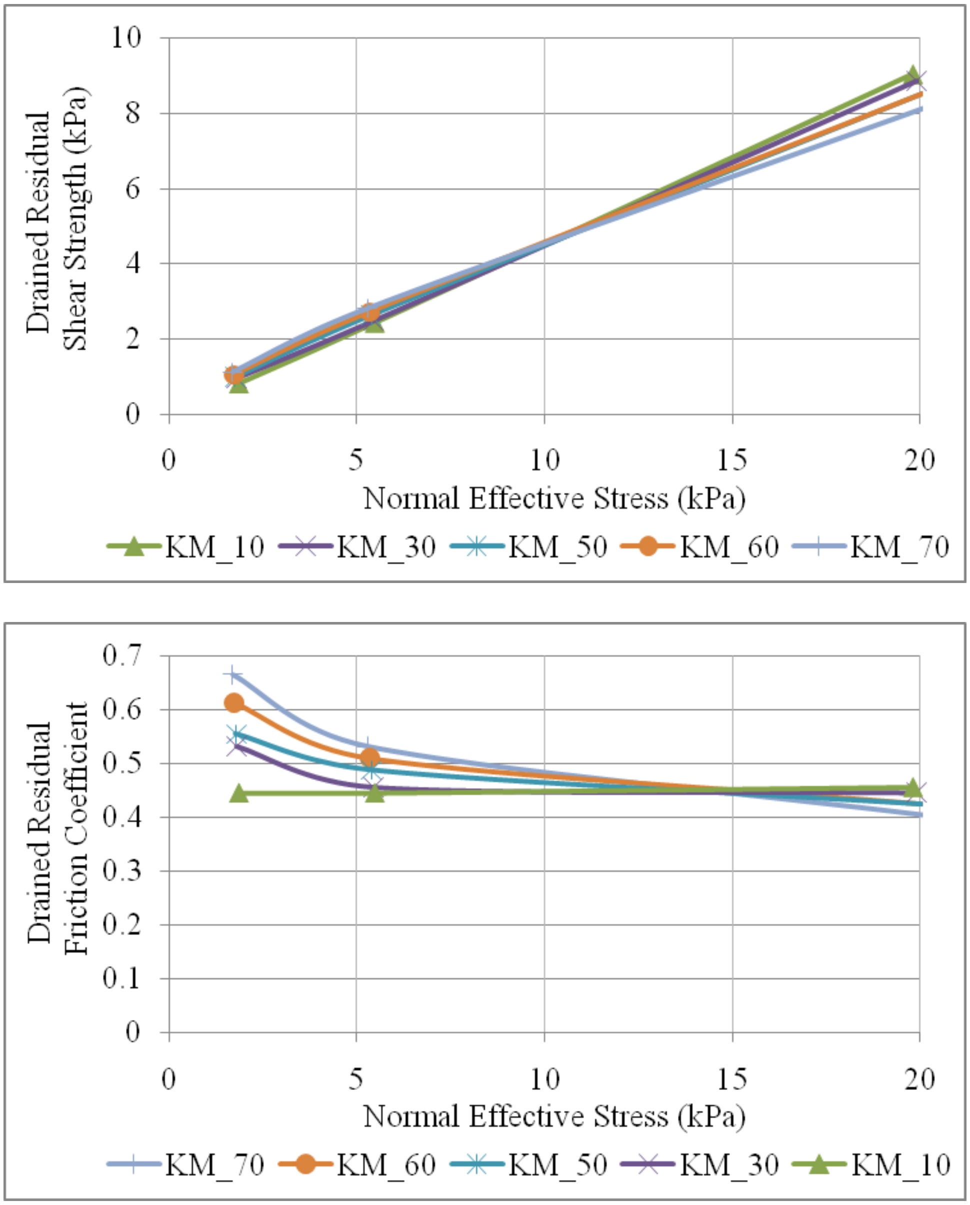


Figure 6.2 Summary of Test Results on Pure Clay and Sand (Internal)



* KM_70: 70 percent Kaolinite / 30 percent Sand (The proportion of each dry weight to the total dry weight)

Figure 6.3 Summary of Test Results on Soil Mixtures (Internal)

6.3 Data Analysis

6.3.1 Effect of Interface on the drained residual shear strength of soils

The measured drained residual strengths from the interface tests using the rough pipeline coating are identical with the internal drained shear strength of the soil, meaning that the pipeline coating roughness (95μm) exceeds the critical value so that only shear failure within the soil specimens instead of interface sliding could occur. For two tests conducted using the smooth interface with marine clay from source BC2, the coating efficiencies are about 96 percent, meaning that the roughness (5μm) is near to critical roughness for the clay from source BC2.

For the sand, kaolinite, and sand-kaolinite mixtures, the coating efficiency against the smooth interface seems to be independent of normal effective stress level in these tests (Figure 6.4). In mixtures with high clay contents (70 percent), the residual interface shear strength approximates to the residual shear strength of soil itself, while the presence of smooth interface induces sliding shear so as to give a lower residual strength in mixtures with low clay contents (10 percent). The interface shearing resistance depends on the roughness of interface material and clay mineralogy (Lemos and Vaughan 2000). Test results on smooth interface are summarized in Table 6.3 in terms of coating efficiency.

Table 6.3 Summary of Tests Results on Smooth Interface

No.	Test ID	Kaolinite (%)	Monterey #30 sand (%)	Failure type	Smooth interface				Coating efficiency
					N stress at failure (kPa)	Residual secant friction	S stress at failure (kPa)	Coefficient of friction	
16	KM_10_2	10	90	Internal	1.9	15	0.5	0.27	0.60
17	KM_70_2	70	30	Internal	1.7	31.5	1.1	0.61	0.92
18	KM_10_6	10	90	Combination	5.8	15	1.5	0.27	0.60
19	KM_70_6	70	30	Internal	5.5	24	2.4	0.45	0.84
20	KM_10_20	10	90	Combination	21.0	15	5.6	0.27	0.59
21	KM_70_20	70	30	Internal	20.2	22	8.2	0.40	1.00
22	M_2	0	100	Internal	1.8	24	0.8	0.45	0.80
23	M_6	0	100	Internal	5.5	24	2.4	0.45	0.80
24	M_20	0	100	Internal	20.1	23	8.5	0.42	0.77
25	K_2	100	0	Internal	1.7	31.5	1.1	0.61	0.84
26	K_6	100	0	Internal	5.4	25	2.5	0.47	0.81
27	K_20	100	0	Internal	20.5	20	7.5	0.36	0.82

Combination = interface sliding and shear deformation of the soil proceed simultaneously

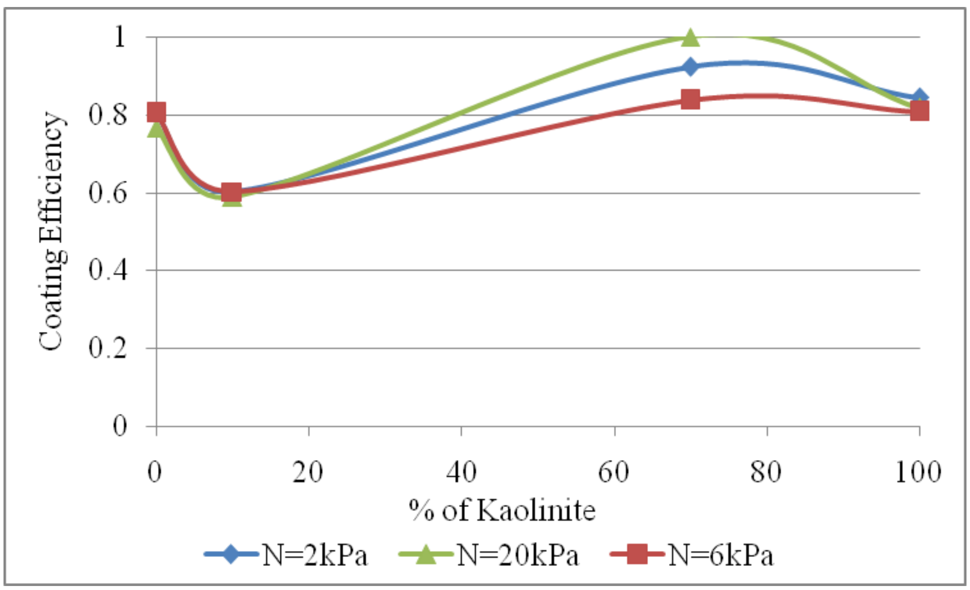


Figure 6.4 Variation of Coating Efficiency with Clay Contents

For the sand, kaolinite, and sand-kaolinite mixtures, visual observations of the failure surface in the tests performed on the soil mixtures indicate that the failure plane is created inside the soil specimen and not at the interface between the soil and the smooth pipeline coating. In some tests, it was hard to determine by observation whether failure occurred at the interface or in the middle of soil. However, results in Figure 6.5 indicate that measured shear strength at the interface is smaller than that of soils even though the failure surface was created at the middle of clay, meaning that partial sliding could occur when soils tested against a smooth, hard interface and thus leading to a reduction in the measured residual interface strength (Skinner 1969). For the tests using marine clay from source BC2 with smooth interface, the soil slid over the interface completely and clear interface surface was observed after the test, but the measured residual interface strength was equivalent to the residual shear strength of soils. This difference could be related to the clay mineralogy and relative particle size of soil specimen.

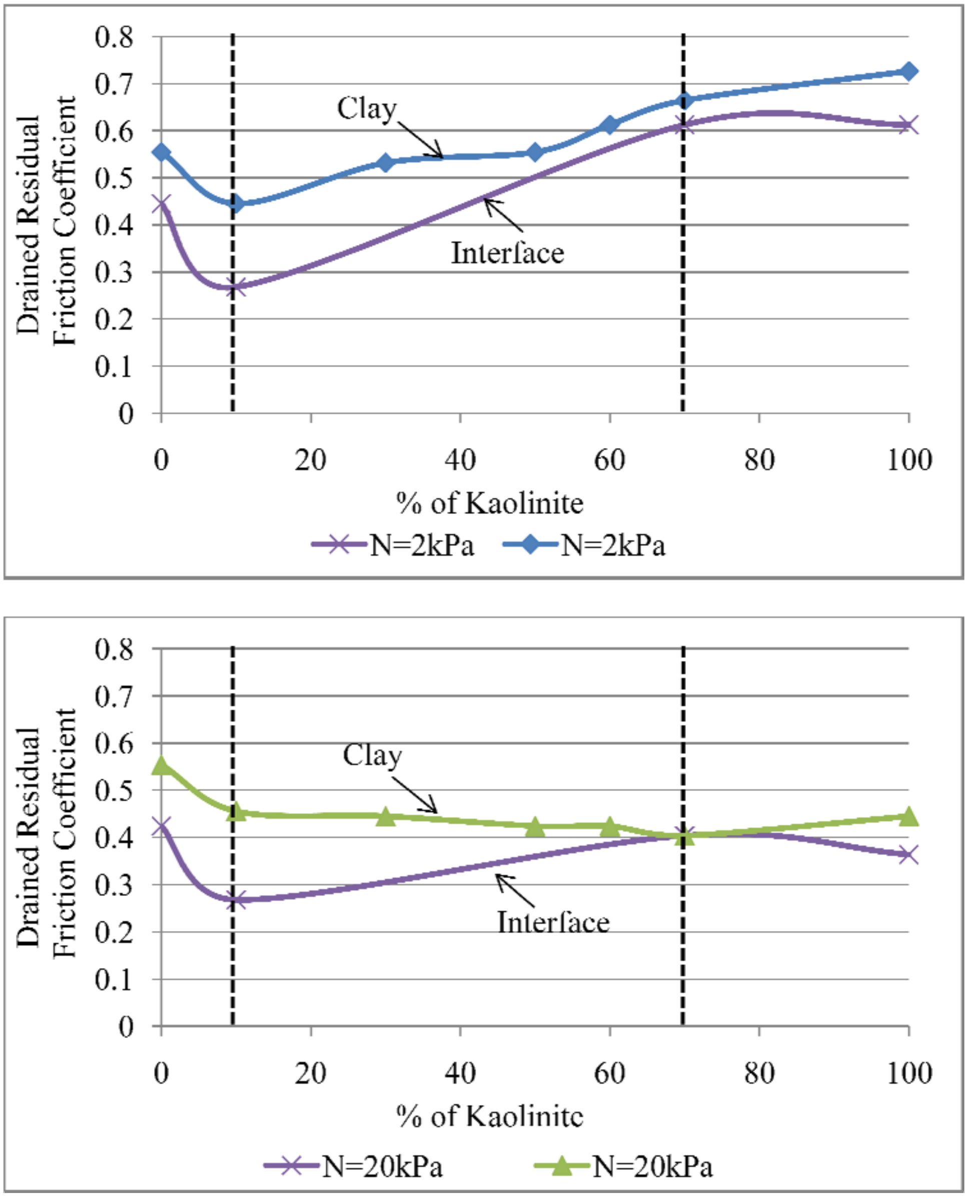


Figure 6.5 Variation of Friction Coefficient with Clay Contents and Interface

6.3.2 Effect of Over-Consolidation Ratio

To investigate the effect of OCR, additional tests are performed using kaolinite. Soil sample is spread on the geotextile with 2mm in thickness and fully consolidated under 6 kPa, 10 kPa, and 20 kPa, before submerging the interface to achieve three different OCR values. The pre-pressure is removed and then the tests are conducted under the effective normal stress of 2 kPa. The test results are presented in Figure 6.6.

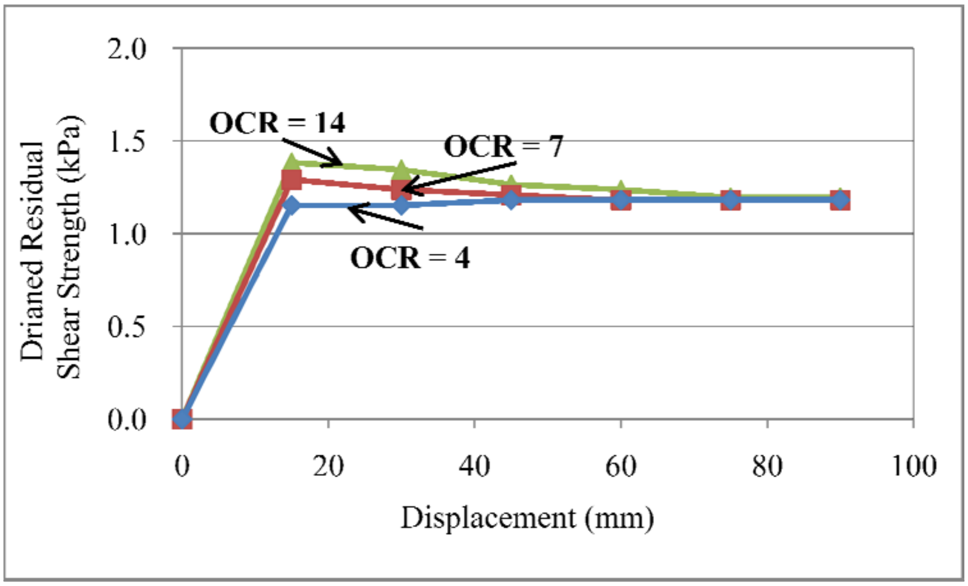
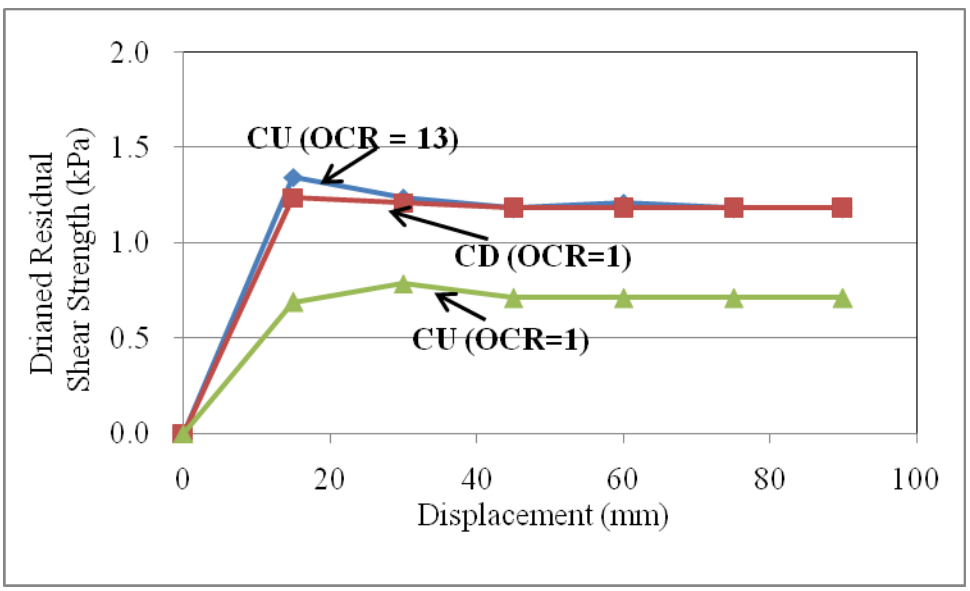


Figure 6.6 Residual Shear Strength with Displacement

The method of specimen preparation and stress history do not affect the drained residual shear strength (Figure 6.6). The results shown are consistent with previous studies (Bishop, Green et al. 1971). Note that the clay exhibits brittleness slightly when sheared after overconsolidation because the clay is not in a remolded state anymore. Therefore, the drained peak strength of remolded soil does not necessarily represent the in situ strength when the soil is sheared initially.

6.3.3 Effect of Loading Rate

For the undrained shear test, the specimen is prepared using kaolinite and sheared at 2 kPa of effective normal stress. The tests are conducted without drainage material at a high loading rate. The clay sheared internally and the failure surface is located in the middle of the soil by observation after the tests. The measured undrained shear strength for the normally consolidated clays is lower than the drained residual shear strength (Figure 6.7). However, the measured undrained shear strength of the heavily overconsolidated kaolinite is similar to the drained residual shear strength at the same effective normal stress.



CU: Consolidated-Undrained Shear Test , CD: Consolidated-Drained Shear Test

Figure 6.7 Variation of Residual Shear Strength with Displacement and OCR

The undrained shear strength of normally consolidated clays can be expressed in terms of the c/p ratio*. If the clay is overconsolidated, the undrained shear strength is normalized with respect to the preconsolidation stress. Ladd, *et al* (1977) showed that the ratio of c/p ratio for overconsolidated clays to c/p ratio for normally consolidated clays is approximately equal to the OCR(overconsolidation ratio) to the 0.8 power. The c/p ratio of both normally consolidated clay and highly overconsolidated clay in Figure 6.7 is about 0.35 and 2.7, respectively. Based on this calculation, the undrained shear strength of clay (OCR=13) is expected to be about 5.4 kPa. However, the measured undrained shear strength of highly overconsolidated clay is equivalent to the drained shear strength in this test. One logical explanation for this result is that the drainage path could be created along between clay and geomembrane, thus leading to drained condition at the top of the specimen.

For the creep test, since the standard test showed the residual angle of 36 ° (Table 6.2), the tilt table was left at an angle of 34° for overnight. The next day, the soil specimen showed no noticeable displacement. Afterward, the failure occurred at 36 ° as tilting the table with standard time interval, meaning that the drained residual strength does not change with time. These test results suggest that the drained residual shear strength will be measured by tilt table device.

* The ratio of undrained shear strength of clay to effective normal stress.

Chapter 7 Discussion

7.1 Failure Mechanism

Three failure modes are observed in these tests: (1) internal failure at the middle of the soil; (2) partial sliding at the interface (combination failure); (3) full sliding at the interface (interface failure), as shown Figures 7.1 and 7.2. The failure surface occurs internal to the soil specimen for the tests aimed at measuring the drained residual strength of the soil. For the tests aimed at measuring residual shear strength at an interface, the clean interface surface is observed only in the tests performed with the marine clays on the smooth interface (Figure 7.2). In the other tests, such as undrained tests and tests using sand and soil mixtures on the smooth interface, the tests seems to involve failure within the soil itself by observation, but the measured residual strengths indicate that failure occurs by partial sliding at the interface. This mechanism could be related to the effective normal stress level, relative particle size, clay contents, and clay mineralogy.

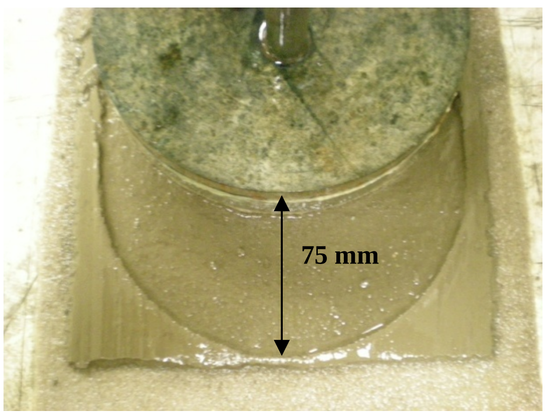


Figure 7.1 Failure Mechanism at Residual Strength (Internal Failure)

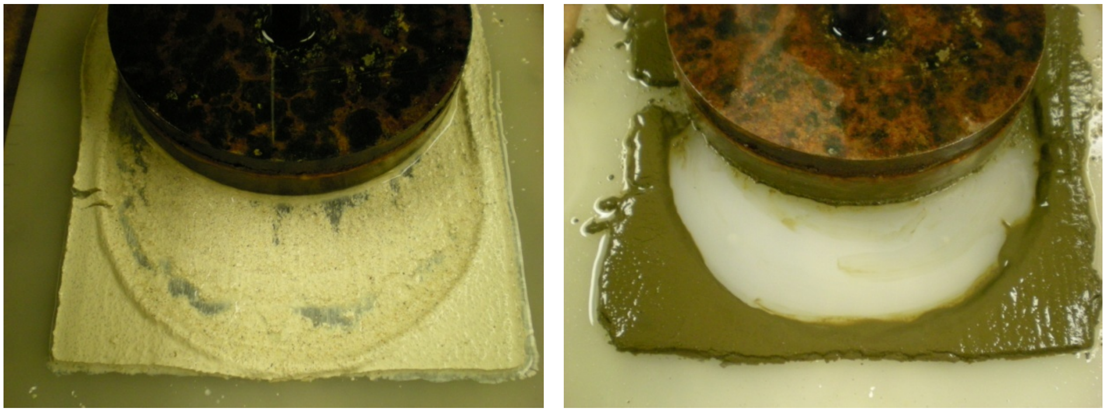


Figure 7.2 Failure Mechanism at Residual Strength (Combination / Interface Failure)

These mechanisms can be identified by observing failure surface during the tests. The drained residual conditions are obtained after about 30 to 50 mm of total displacement in all tests. The displacement required to achieve the residual conditions for soil mixtures is less than it is when shearing the marine clays. This result could be related to the clay mineralogy and clay contents. Figure 7.2 exhibits an example of load-displacement curve for the test using marine clay from source BC1 (curves for the test using kaolinite, sand are provided in Appendix E). This soil is classified as extremely high plasticity soil. Most previous studies have indicated that the higher plasticity soils exhibit more strain softening, preferred particle orientation when sheared to large strain, and low drained residual shear strength (Lupini, Skinner et al. 1981)*. However, the load-displacement curves do not exhibit noticeable peak drained shear as shown in Figure 7.2. The main reason is that the soil specimens were fully softened state at the beginning of

* All of tests were performed under an effective normal stress higher than 100kPa.

the tests by remolding the samples and thus failure mechanism is not related to the clay particle reorientation in the direction of shear. The low effective normal stresses used in these tests could be another reason that the difference between the peak and the residual shear strength is negligible.

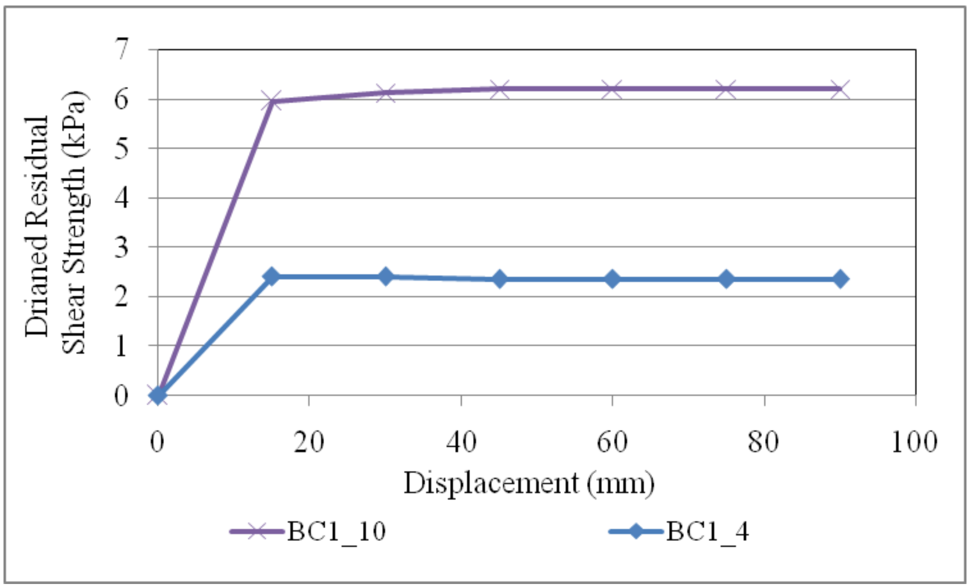


Figure 7.3 Typical Load-Displacement Curves

7.2 Effect of Normal Stress (Nonlinearity of Failure Envelope)

Most tests exhibit a continuous drop in residual friction coefficient with increasing effective normal stress, which is the most noticeable at very low effective normal stresses less than 5 kPa (Figure 7.4), while the cohesionless soils show a constant residual secant friction angle. As shown in Figure 7.5, the coefficient of friction for the drained residual shear strength decrease as the effective normal stress increases both for the interface and for the internal strength. The residual secant friction angle decreases as the normal stress increases. This type of tendency in failure envelope for the drained residual strength is general in cohesive soils (Skempton 1985). The failure envelope for the soils from source BC1, BC2, and BC3 curves down slightly, while that for the Kaolinite exhibits significant curvature under effective normal stress lower than 5 kPa. For the Monterey #30 sand, the failure envelope is essentially linear with increasing effective normal stresses.

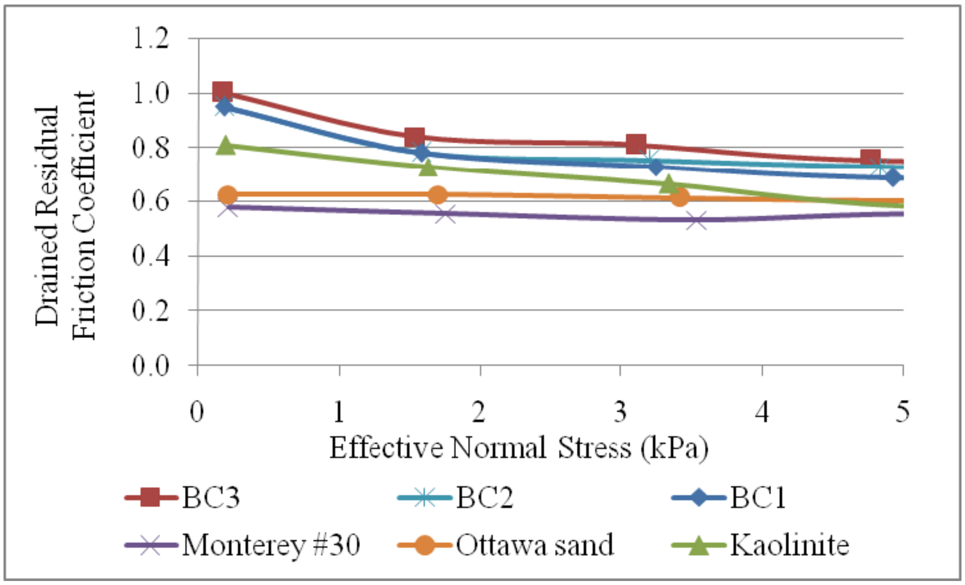


Figure 7.4 Variation of Friction Coefficient with Effective Normal Stress (Internal)

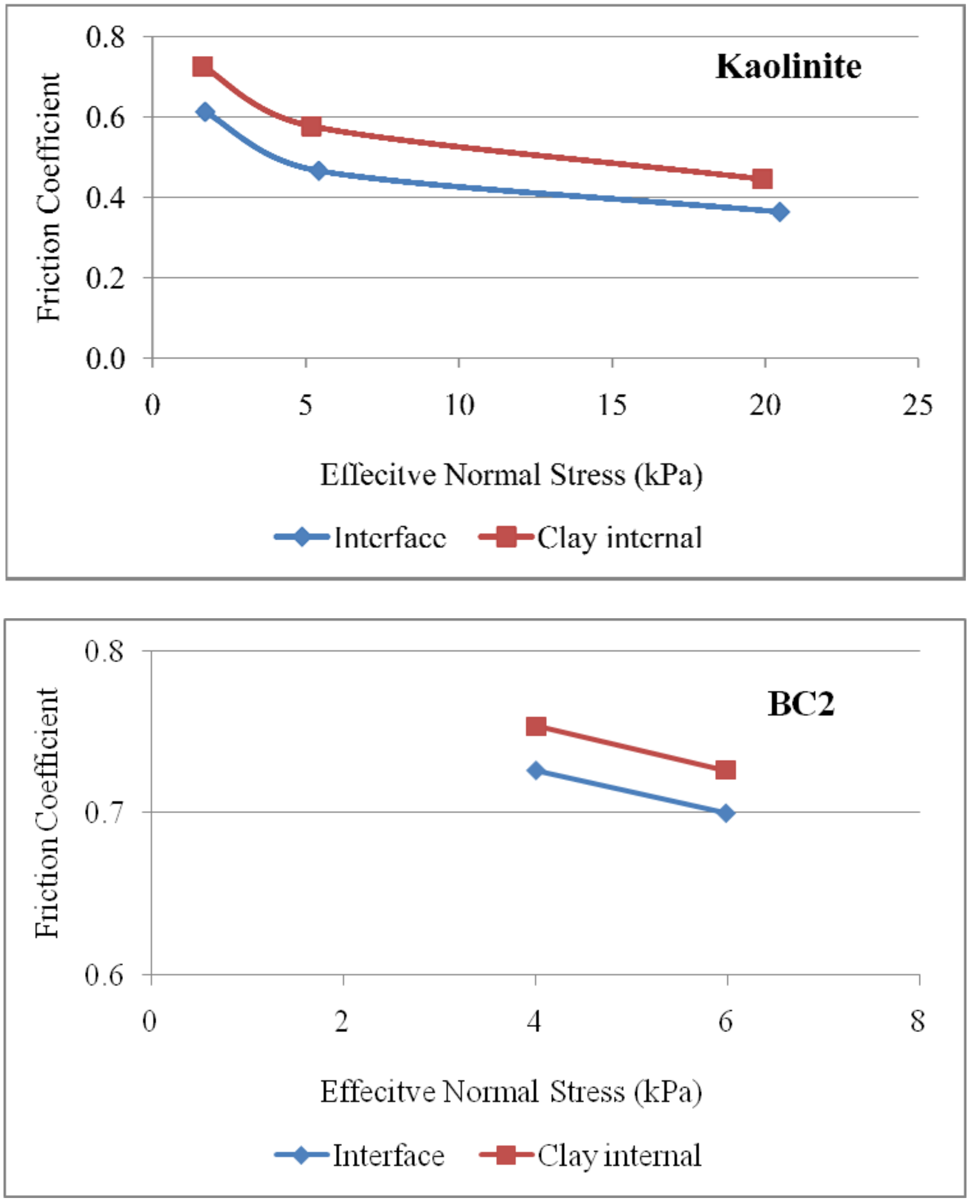


Figure 7.5 Variation of Friction Coefficient with Effective Normal Stress (Smooth interface)

7.3 Effect of Soil Compositions

The differences in the drained residual shear strength of the clays from source BC1, BC2, and BC3 are not very significant under given normal stress ranges in comparison to that of the Kaolinite and Monterey #30 sand (Figure 7.6). Since the drained residual shear strength of cohesive soils will be affected by the combinations of various factors such as the clay content, the plasticity of soil, the size distribution of soil particles, and the clay mineralogy, there are no simple correlations between the residual shear strength and index properties of soils, especially for natural marine clays (Lupini, Skinner et al. 1981).

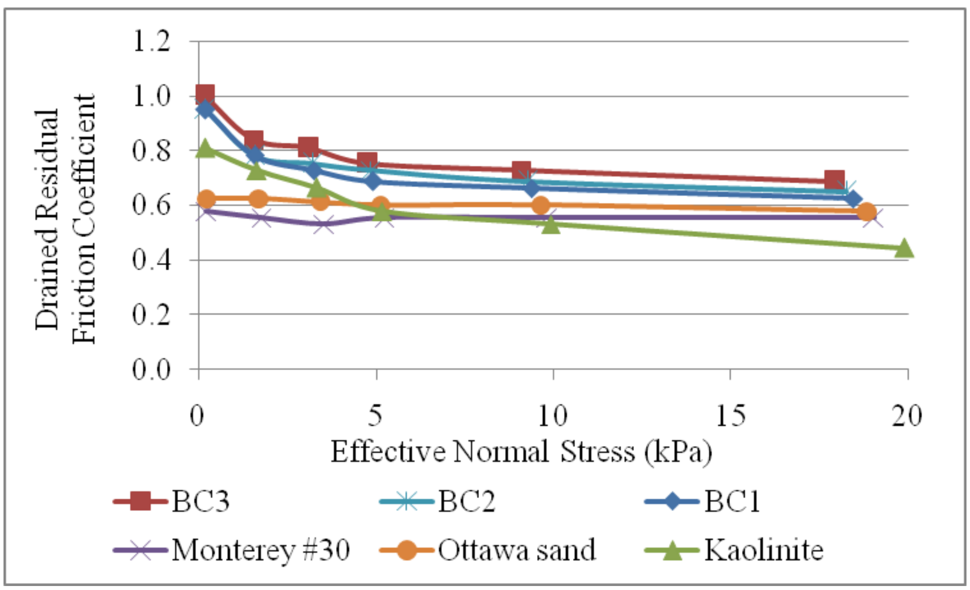


Figure 7.6 Variation of Friction Coefficient with Effective Normal Stress

Note that all of the tests in the previous studies were performed at the effective normal stress higher than 100kPa. In this point of view, there are noticeable points about factors affecting the drained residual shear strength of soils at low effective normal stress comparing with that at high effective normal stress, as shown in Figure 7.6.

1. The soil having higher plasticity shows the higher friction coefficient under the effective normal stress ranges of 0.25 to 20 kPa. The drained residual shear strength of clay is slightly higher than that of silt. For the same soil type having the same clay contents, the clay having higher activity exhibits the higher drained residual shear strength.
2. The clay mineralogy and the proportions of clay to massive particles are also important factors on residual friction angle (Kenney 1967; Kenney 1977). The kaolinite has clay of 100 percent, but shows the lowest drained residual shear strength, meaning that the clay contents alone is not proportional to the drained residual shear strength.

The measured drained residual shear strength of soils from source BC1, BC2, and BC3 are higher than that of sands. For the kaolinite, under very low effective normal stress less than 5 kPa, the measured value is also higher than that of sand. The coefficient of friction proposed by Stark and Eid (1994) for the clays having the same range of liquid limit with soils from source BC1, BC2, and BC3 (100 to 140 percent) is between 0.15 and 0.2 under effective normal stresses ranging from 100 to 700 kPa. These results are apparently contributed to the magnitude of effective normal stresses. Therefore, shear test results conducted under higher stress levels must not be extrapolated to lower effective normal stresses.

7.4 Effect of Physico-Chemical Change in Pore Fluid

Table 7.1 Values of Activity for some Clay Minerals (Skempton 1953)

Mineral	Activity	Reference
Quartz	0	von Moos (1938)
Calcite	0.18	von Moos (1938)
Mica	0.23	von Moos (1938)
Kaolinite	0.33(0.46)	Northey(1950)(Samuels(1950))
Illite	0.9	Northey (1950)
Ca-montmorillonite	1.5	Samuels (1950)
Na-montmorillonite	7.2	Samuels (1950)

Based on the index properties and size distribution of clays from source BC1, BC2, and BC3, the activities for all clays range from 5.6 to 8.8. It is expected that the clay samples at the project site contain sodium montmorillonite. It is well known that an even small amount of clays can be attributed to the important role in determining residual shear strength, especially in case of the clay minerals having high activity. The activity of clays decreases with increasing in NaCl concentration in pore fluid which causes decrease in diffuse double layer thickness thus leading to flexible particles of low physical strength when it is subjected to the high effective normal stresses (Kenney, Moum et al. 1967; Olson 1974). Increasing in NaCl concentration in pore fluid causes change in clay structure from dispersive to flocculate conditions, thus leading to increase an internal friction angle and decrease a liquid limit (Moore 1991; Tiwari, Tuladhar et al. 2005). Ramiah *et al.* (1970) reported that the residual friction angle decreased from 33 to 28 degrees by changing pore water chemistry from high NaCl to low NaCl conditions. However, the decrease in the value of shear strength components will depend on the clay mineralogy and applied normal stress level.

Table 7.2 Variation of Friction Coefficient with Salinity of Pore Fluid

Test ID	Soil Source	Soil Classification	Test type	Thickness (mm)	Interface	Coefficient of friction	
						Salt water	Tap water
BC1_2	BC1	ME	CD	2	GT	0.78	0.75
BC2_2	BC2	ME	CD	2	GT	0.78	0.78
BC3_2	BC3	CE	CD	2	GT	0.84	0.78
BC1_20	BC1	ME	CD	2	GT	0.62	0.61
BC2_20	BC2	ME	CD	2	GT	0.65	0.61
BC3_20	BC3	CE	CD	2	GT	0.69	0.61

In order to evaluate the effect of leaching the marine clays having high salt concentration in pore fluid with the tap water under low effective normal stress, six tests are performed with clays from source BC1, BC2, and BC3 mixing with tap water and tested in a tap water bath. As shown in Table 7.2, there have been a slow reduction in the salinity of pore fluid, and this have resulted in decrease of residual shear strength and this effect will increase with time. Since the pore water chemistry affects the residual shear strength of cohesive soils, it is very important to use an appropriate chemical solution in the water bath thus leading to minimize the physico-chemical effect on residual shear strength during testing.

7.5 Effect of Normal Stress with Clay Contents

Increasing normal stress will cause a reduction of the residual friction angle in sliding and a denser packing of the rotund particles simultaneously. Therefore, these two mechanisms have to be considered to determine the total effect of normal stress. Since the failure envelope of Monterey #30 sand intersects that of Kaolinite at the effective normal stress of 6kPa, the residual shear behavior of the mixtures may change as the applied effective normal stresses increase, over 6kPa. The shear strength of cohesive soils can be represented by a cohesion intercept and a coefficient of internal friction (Jakobson 1953). These two components of shear strength for each soil mixture are summarized in Table 7.3.

Table 7.3 Variation of Friction Angle with Effective Normal Stress Level

No.	Soil	Liquid Limit(%)	Plasticity Index(%)	N Stress at Horizontal (kPa)	Residual Friction Coefficient	Best Linear Envelope (2~6kPa)		Best Linear Envelope (6~20kPa)	
						c (kPa)	Ø	c (kPa)	Ø
1				2.01					
2	Monterey	N/A	N/A	5.98	0.55	0	29	0	29
3				21.79					
4	10%			2.01	0.45				
5	Kaolinite/90%	6	2	5.98	0.45	0	24	0	25
6	Monterey			21.79	0.46				
7	30%			2.01	0.53				
8	Kaolinite/70%	17	7	5.98	0.46	0.2	23	0.1	24
9	Monterey			21.79	0.45				
10	50%			2.01	0.55				
11	Kaolinite/50%	28	12	5.98	0.49	0.2	24	0.5	22
12	Monterey			21.79	0.42				
13	60%			2.01	0.61				
14	Kaolinite/40%	34	15	5.98	0.51	0.3	25	0.6	21
15	Monterey			21.79	0.42				
16	70%			2.01	0.66				
17	Kaolinite/30%	39	17	5.98	0.53	0.3	25	0.9	20
18	Monterey			21.79	0.40				
19				2.01	0.73				
20	Kaolinite	56	25	5.98	0.58	0.4	27	1	22
21				21.79	0.45				

The decrease in friction angle with increasing clay contents is common for cohesive soils at high effective normal stresses (Lupini, Skinner et al. 1981; Skempton 1985; Collotta, Cantoni et al. 1989). As shown in Table 7.3, for the effective normal stress higher than 6 kPa in which the rotund particles govern a shearing mechanism, the clay particles could form a shear zone between well-dispersed rotund particles and prevent the rotund particles from interlocking each other and thus leading to decrease in residual friction angle with increasing clay contents. However, the data in Table 7.3 also indicate that the estimated friction angle at normal stresses ranging from 2 to 6 kPa increases with increasing clay contents. It is apparent that the main reason for this difference in trends is the effective normal stress level. These results indicate that the correlations for predicting the residual shear strength of cohesive soils cannot be extrapolated to the low effective normal stresses.

As shown in Figure 7.7, under the effective normal stress of 20 kPa, the soil mixtures exhibit the similar correlations to those reported in the literature. Most previous studies for drained residual shear strength of cohesive soils have concluded that the drained residual friction coefficient decreases as clay contents increase (Bishop, Green et al. 1971; Lupini, Skinner et al. 1981; Skempton 1985)[*]. However, the soil mixtures exhibit the reverse behavior at the effective normal stresses less than 6kPa.

Interesting finding is that there is a consistent reduction in the drained residual shear strength of soil mixtures of the lowest clay proportion. One possible explanation is that when the clay content is extremely low (10%), the residual strength may be consistent with the effective normal stress like pure sand, but the small amount of clay particles perhaps prevent the sand particles from interlocking each other and thus leading to a reduction in shearing resistance even though there is no preferred orientation of the clay particles.

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[*] All tests were performed under the effective normal stresses higher than 100kPa.

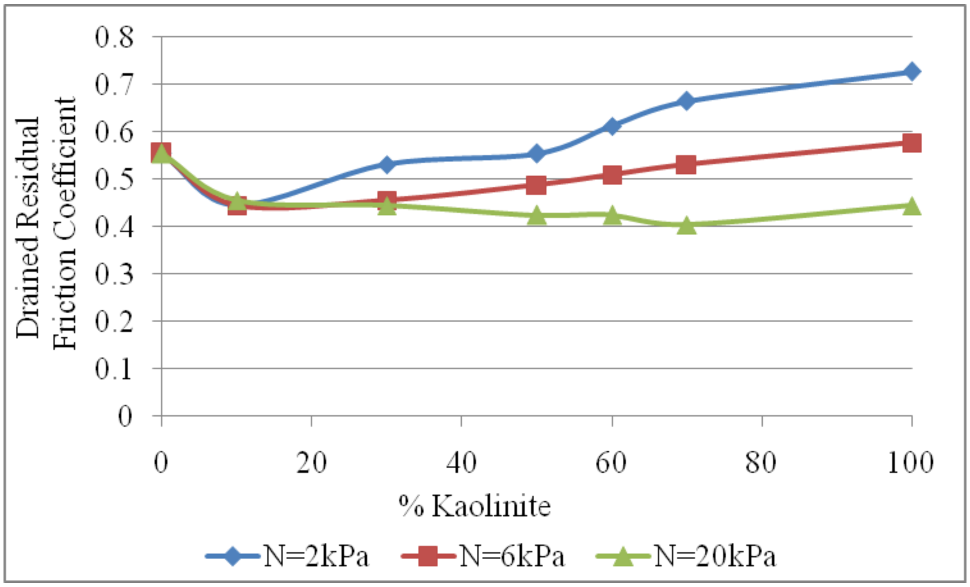


Figure 7.7 Variation of Friction Coefficient with Clay Contents

Chapter 8 Conclusions

A tilt table method is described for measuring the drained residual shear and interface strength of soil. The following conclusions can be drawn about the shearing behavior of soils at low effective normal stresses based on both the test results and the previous studies. A series of 74 tests are conducted on 3 different types of interface with 11 types of soils under six different levels of normal stresses ranging from 0.25 to 20 kPa.

1. The tilt table method can be used to characterize the drained residual shear strength of soils and at interfaces at low effective normal stresses and also provide high repeatability of test results (Appendix C)
2. The residual condition is mobilized at less than 50mm of displacement along the interface for all tests and there is no significant drop from peak to residual strength.
3. The drained residual shear strength both for the interface and for the soils is not affected by the overconsolidation ratio.
4. The drained residual shear strengths for the interfaces are less than the drained residual shear strengths of soils. The drained residual strength of interface depends on the roughness of interface, clay mineralogy.
5. Increasing effective normal stress leads to changes in failure mechanism and the residual shear strength of cohesive soils and at interfaces.

6. Cohesionless soils exhibit a constant residual secant friction angle regardless of effective normal stress levels.
7. The effect of the salinity of pore fluid on the drained residual shear strength is significant at low effective normal stress. This is strongly related to both the clay mineralogy and the magnitude of effective normal stresses.
8. Clay mineralogy and clay contents together with the magnitude of effective normal stress are the most important factors to estimate the drained residual shear strength of cohesive soils.
9. It is not possible to extrapolate the empirical correlations at higher effective normal stresses to lower effective normal stresses.

Marine clays used in this study include about 10 to 15 percent of clay particles and exhibit much higher drained shear strength than that of sand-kaolinite mixtures. Based on activity, it is expected that marine clays contain bentonite. For sand-kaolinite mixtures, the sample, having 10 percent of clay exhibit the lowest drained residual shear strength, meaning that clay contents alone does not have an effect on the residual strength. In order to investigate the effect of clay contents with clay mineralogy on the residual shear strength at low effective normal stresses, it is needed to perform a series of tests with soil mixtures using different types of clay mineral, such as bentonite and illite. This result could contribute to understanding the reason that soft marine clays exhibit higher residual shear strength than that of sand at low effective normal stresses.

Appendix

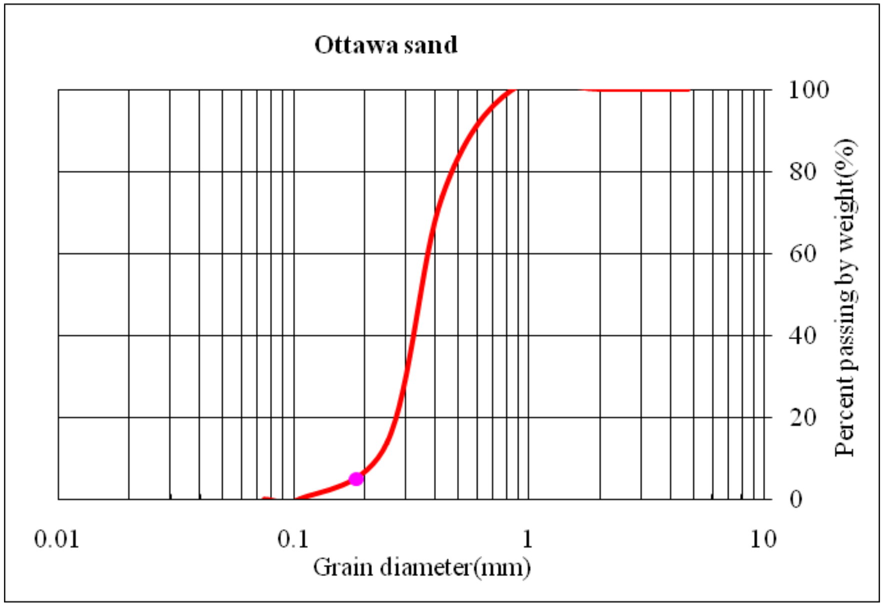
Appendix A: Raw Data

NO.	Test ID	Soil Source	Soil Classification	Pore water	Test type	Thickness (mm)	Interface	Target pressure (kPa)	N stress at horizontal(kPa)	OCR	Weights used	Failure type	Angle history	N stress at failure (kPa)	Residual angle	S stress at failure (kPa)	Coefficient of friction	w(%)	w(%) at failure plane	Observations	time for consolidation	time interval
1	BC1_0	BC1	ME	Salt water	CD	2	95mu	0.25	0.25	1.4	D6A+250	Internal	43.5/43.5	0.2	43.5	0.2	0.9	90		With thick GT, Air bubbles come out and sustain over 60 deg at first undrain test	Overnight	5
2	BC1_2	BC1	ME	Salt water	CD	2	95mu	2	2.01	1.3	D6S+2000	Internal	39/39/37/38/38	1.6	38	1.2	0.8	91		Failure plane is very rough, suspect that GT protrude into the clay and affect interanl strength	60min	5
3	BC1_4	BC1	ME	Salt water	CD	2	95mu	4	4.01	1.2	D6S+5000	Internal	37/37/36/36/36	3.2	36	2.4	0.7	102		Pay attention to undrain failure	20min	5
4	BC1_6	BC1	ME	Salt water	CD	2	95mu	6	5.98	1.2	D6S+8000	Internal	34.5/34.5/34.5/34.5/34.5/34.5	4.9	34.5	3.4	0.7	100		suspect that failure occurs b/w porous stone and clay.	60	5
5	BC1_10	BC1	ME	Salt water	CD	2	GT	10	11.25	1.2	D6S+16000	Internal	32/33.5/33.5/33.5	9.4	33.5	6.2	0.7	90		Drain fail, care about the possibility that failure occurs b/w porous stone and eccentricity increases against the smaller area.	60	5
6	BC1_20	BC1	ME	Salt water	CD	3	95mu	20	21.79	1.2	D6S+32000	Internal	31.5/32/32/32/32	18.5	32	11.5	0.6	90			60	5
7	BC2_0	BC2	ME	Salt water	CD	2	GT	0.25	0.25	1.4	D6A+250	Internal	45/43.5/43.5/43.5	0.2	43.5	0.2	0.9	142				
8	BC2_2	BC2	ME	Salt water	CD	2	GT	2	2.01	1.3	D6S+2000	Internal	42/40/39.5/39/38/38	1.6	38	1.2	0.8	138		Negative p.w.p? , Need creep test to estimate reasonable residual friction angle.	60min	5
9	BC2_4	BC2	ME	Salt water	CD	2	GT	4	4.01	1.3	D6S+5000	Internal	36.5/37/37/37/37	3.2	37	2.4	0.8	168		Failure occurs after waiting almost 15min (Need more reasonable time interval)	60min	5
10	BC2_6	BC2	ME	Salt water	CD	2	GT	6	5.98	1.2	D6S+8000	Internal	33/35/37.5/36.5/36.5/36	4.8	36	3.5	0.7	167		Filter changes after this test. The loading plate is dragging down the clay and hill at the end of the plate, w/c : 167%	60min	5
11	BC2_10	BC2	ME	Salt water	CD	2	GT	10	11.25	1.2	D6S+16000	Internal	33/33/34.5/34.5/34.5	9.3	34.5	6.4	0.7	167				
12	BC2_20	BC2	ME	Salt water	CD	3	GT	20	21.79	1.2	D6S+32000	Internal	32/33/33/33/33	18.3	33	11.9	0.6	167				
13	BC3_0	BC3	CE	Salt water	CD	2	GT	0.25	0.25	1.4	D6A+250	Internal	57/42/45/45/45/45	0.2	45	0.2	1.0	162		After fully consolidation, slow and rapid loading have almost the same residual strength.	Overnight	5
14	BC3_2	BC3	CE	Salt water	CD	2	GT	2	2.01	1.3	D6S+2000	Internal	43.5/40/40/40/40	1.5	40	1.3	0.8	164		With thick GT (-> thick GT looks like protrude into the clay and affect the result significantly, not gonna use anymore)	60	5
15	BC3_4	BC3	CE	Salt water	CD	2	GT	4	4.01	1.3	D6S+5000	Internal	42/39/40/39/39	3.1	39	2.5	0.8	164		With thick GT, rapid loading for first failure, air bubbles come out right before failure occurs.	Overnight	5
16	BC3_6	BC3	CE	Salt water	CD	2	95mu	6	5.98	1.3	D6S+8000	Internal	34.5/37/37/36/37	4.8	37	3.6	0.8	164		With thick GT	60	5
17	BC3_10	BC3	CE	Salt water	CD	2	GT	10	11.25	1.2	D6S+16000	Internal	31.5/35/36/36/36/37/36	9.1	36	6.6	0.7	164		rapid loading for first failure	Overnight	5
18	BC3_20	BC3	CE	Salt water	CD	3	GT	20	21.79	1.2	D6S+32000	Internal	26/31.5/34.5/34.5/34.5	18.0	34.5	12.3	0.7	162		rapid loading for first failure, need to adjust load eccentricity, drain residual angle will be smaller than measured one.	120	5
19	K_0	Kaolinite	CH	Tap water	CD	2	GT	0.25	0.25	1.3	D6A+250	Internal	48/45/43.5/42/40/39/39/39	0.2	39	0.2	0.8	77		shear rate looks like not gonna affect the residual strength. (the results are from repeated rapid loading..)	120	5
20	K_2	Kaolinite	CH	Tap water	CD	2	GT	2	2.01	1.2	D6S+2000	Internal	38/37/36/36/36	1.6	36	1.2	0.7	77			Overnight	5
21	K_4	Kaolinite	CH	Tap water	CD	2	GT	4	4.01	1.2	D6S+5000	Internal	31.5/33.6/33/33.6/33.6	3.3	33.6	2.2	0.7	77		Undrain vs Drain test, from now on tab water will be used.	120	5
22	K_6	Kaolinite	CH	Tap water	CD	2	GT	6	5.98	1.2	D6S+8000	Internal	20/29/30/30	5.2	30	3.0	0.6	76		Applying rapid loading to intend undrain test at the first stage, and then doing test with full drainage	Overnight	5
23	K_10	Kaolinite	CH	Tap water	CD	2	GT	10	11.25	1.1	D6S+16000	Internal	22/27/28/28/28/28	9.9	28	5.3	0.5	77		looks more reasonable.	120	5
24	K_20	Kaolinite	CH	Tap water	CD	3	GT	20	21.79	1.1	D6S+32000	Internal	18/24/24/24/24	19.9	24	8.9	0.4	77		failure occurs very slowly.	Overnight	5
25	M_0	Monterey #30	SP	Tap water	CD	2	GT	0.25	0.25	1.1	D6A+250	Internal	29/29	0.2	29	0.1	0.6	27				
26	M_2	Monterey #30	SP	Tap water	CD	2	GT	2	2.01	1.1	D6S+2000	Internal	29/29	1.8	29	1.0	0.6	27				
27	M_4	Monterey #30	SP	Tap water	CD	2	GT	4	4.01	1.1	D6S+5000	Internal	29/29	3.5	29	1.9	0.6	27				
28	M_6	Monterey #30	SP	Tap water	CD	2	GT	6	5.98	1.1	D6S+8000	Internal	29/29	5.2	29	2.9	0.6	27				
29	M_10	Monterey #30	SP	Tap water	CD	2	GT	10	11.25	1.1	D6S+16000	Internal	29/29	9.8	29	5.5	0.6	27				
30	M_20	Monterey #30	SP	Tap water	CD	2	GT	20	21.79	1.1	D6S+32000	Internal	29/29	19.1	29	10.6	0.6	27				
31	S_0	Ottawa Sand	SP	Tap water	CD	2	GT	0.25	0.25	1.2	D6A+250	Internal	33.5/33/33.5/32/32/32	0.2	32	0.1	0.6	27				
32	S_2	Ottawa Sand	SP	Tap water	CD	2	GT	2	2.01	1.2	D6S+2000	Internal	32/32/32/32/32/32	1.7	32	1.1	0.6	27				
33	S_4	Ottawa Sand	SP	Tap water	CD	2	GT	4	4.01	1.2	D6S+5000	Internal	31.5/31.5/31.5/31.5/31.5	3.4	31.5	2.1	0.6	27		Rapid failure rate		
34	S_6	Ottawa Sand	SP	Tap water	CD	2	GT	6	5.98	1.2	D6S+8000	Internal	32.5/29.5/31/31/31/31	5.1	31	3.1	0.6	27				
35	S_10	Ottawa Sand	SP	Tap water	CD	2	GT	10	11.25	1.2	D6S+16000	Internal	31/30.5/31/31/31	9.6	31	5.8	0.6	27				
36	S_20	Ottawa Sand	SP	Tap water	CD	3	GT	20	21.79	1.2	D6S+32000	Internal	29/30/30/30	18.9	30	10.9	0.6	27				
37	KDG_3	Kaolinite	CH	Tap water	CD	2	GT		5.98	3.7	D6S+2000	Internal		1.6	36	1.2	0.7					
38	KDG_5	Kaolinite	CH	Tap water	CD	2	GT		11.25	6.9	D6S+2000	Internal		1.6	36	1.2	0.7					
39	KDG_10	Kaolinite	CH	Tap water	CD	2	GT		21.79	13.6	D6S+2000	Internal		1.6	37	1.2	0.8					
40	KUS_1	Kaolinite	CH	Tap water	CU	2	5mu		2.01	#DIV/0!	D6S+2000	Internal						77				
41	KUS_10	Kaolinite	CH	Tap water	CU	2	5mu		21.79	13.4	D6S+2000	Internal	42/38/36/37/36/36	1.6	36	1.2	0.7	77		Should interface failure.. Need thickness adjustment to 1.5mm		
42	KDS_1	Kaolinite	CH	Tap water	CD	2	5mu		2.01	1.2	D6S+2000	Internal	34.5/32/32/31.5/31.5/31.5	1.7	31.5	1.1	0.6	77		Guess partial sliding between interface and soils.		
43	KUR_1	Kaolinite	CH	Tap water	CU	2	95mu		2.01	1.1	D6S+2000	Internal	20/23/20.7/20/19/20.7	1.9	20.7	0.7	0.4	77	73			
44	KUR_10	Kaolinite	CH	Tap water	CU	2	95mu		21.79	12.5	D6S+2000	Internal	34/33/33/30	1.7	30	1.0	0.6	77	64			
45	KDR_1	Kaolinite	CH	Tap water	CD	2	95mu		2.01	1.2	D6S+2000	Internal	34.5/36/36/36/36	1.6	36	1.2	0.7	77				
46	BC2_10	BC2	ME	Tap water	CD	2	5mu		11.25	1.1	D6S+16000	Combination	27/24/21.5/21.5/21.5	10.5	21.5	4.1	0.4	167	144	when marine clay is exposed to tab water. Clay turns into almost liquid.		
47	BC1_2	BC1	ME	Tap water	CD	2	GT		2.01	1.3	D6S+2000	Internal	34.5/37	1.6	37	1.2	0.8					
48	BC2_2	BC2	ME	Tap water	CD	2	GT		2.01	1.3	D6S+2000	Internal	38/39/38/38	1.6	38	1.2	0.8	167	171			
49	BC3_2	BC3	CE	Tap water	CD	2	GT		2.01	1.2	D6S+2000	Internal	36/38/36/36/36	1.6	36	1.2	0.7	164	134			
50	BC1_20	BC1	ME	Tap water	CD	2	GT		21.79	1.2	D6S+32000	Internal	30.7/32/31.5	18.6	31.5	11.4	0.6	100	63			
51	BC2_20	BC2	ME	Tap water	CD	2	GT		21.79	1.2	D6S+32000	Internal	30.7/32/31.5	18.6	31.5	11.4	0.6	167	122			
52	BC3_20	BC3	CE	Tap water	CD	2	GT		21.79	1.2	D6S+32000	Internal	31.5/32/31.5	18.6	31.5	11.4	0.6	164	102			

NO.	Test ID	Soil Source	Soil Classification	Pore water	Test type	Thickness (mm)	Interface	Target pressure (kPa)	N stress at horizontal(kPa)	OCR	Weights used	Failure type	Angle history	N stress at failure (kPa)	Residual angle	S stress at failure (kPa)	Coefficient of friction	Initial w(%)	w(%) at failure plane	Observations
1	KM_0_2	Monterey #30	SP	Tap water	CD	2	5/95mu	2	2.01	1.1	D6S+2000	Internal	22/24/24(29/29/29)	1.8	29	1.0	0.55	27	27(28)	brittle failure
2	KM_10_2	Kaol+Mon #30	ML	Tap water	CD	2	5/95mu	2	2.01	1.1	D6S+2000	Internal	15/15(24/24/24)	1.8	24	0.8	0.45	10	18(17)	brittle failure, need repeat test.
3	KM_30_2	Kaol+Mon #30	CL	Tap water	CD	2	95mu	2	2.01	1.1	D6S+2000	Internal	30.7/29/28/28	1.8	28	0.9	0.53	20	20	
4	KM_50_2	Kaol+Mon #30	CL	Tap water	CD	2	95mu	2	2.01	1.1	D6S+2000	Internal	30/29/29./29	1.8	29	1.0	0.55	35	27	
5	KM_60_2	Kaol+Mon #30	CL	Tap water	CD	2	5/95mu	2	2.01	1.2	D6S+2000	Internal	27/30/29/29/29(31.5)	1.7	31.5	1.1	0.61	42	28	
6	KM_70_2	Kaol+Mon #30	CI	Tap water	CD	2	5/95mu	2	2.01	1.2	D6S+2000	Internal	30.7/33.6/31.5/31.5/31.5(32/33.6/33.6)	1.7	33.6	1.1	0.66	50	40(35)	slow rate of failure
7	**KM_100_2**	**Kaolinite**	**CH**	Tap water	**CD**	**2**	**5/95mu**	**2**	**2.01**	**1.2**	**D6S+2000**	**Internal**	31.5(34.5/36/36/36/36)	**1.6**	**36**	**1.2**	**0.73**	**70**	**60**	
8	KM_0_6	Monterey #30	SP	Tap water	CD	2	5/95mu	6	5.98	1.1	D6S+8000	Internal	24/24(29)	5.2	29	2.9	0.55	27	27	
9	KM_10_6	Kaol+Mon #30	ML	Tap water	CD	2	5/95mu	6	5.98	1.1	D6S+8000	Comb./Internal	15/15/15(24)	5.5	24	2.4	0.45	10	18	This specimen turns into almost liquid after the test
10	KM_30_6	Kaol+Mon #30	CL	Tap water	CD	2	95mu	6	5.98	1.1	D6S+8000	Internal	25.3/24.5/24.5/24.5	5.4	24.5	2.5	0.46	20	18	
11	KM_50_6	Kaol+Mon #30	CL	Tap water	CD	2	95mu	6	5.98	1.1	D6S+8000	Internal	25.3/27/26/26/26	5.4	26	2.6	0.49	35	23	Combination failure, thickness problem?
12	KM_60_6	Kaol+Mon #30	CL	Tap water	CD	2	95mu	6	5.98	1.1	D6S+8000	Internal	27	5.3	27	2.7	0.51	42	27	
13	KM_70_6	Kaol+Mon #30	CI	Tap water	CD	2	5/95mu	6	5.98	1.1	D6S+8000	Internal	20/24/24/24/24(28/28/28)	5.3	28	2.8	0.53	50	37(35)	For first failure, it's undarin condition or normal aspect?
14	**KM_100_6**	**Kaolinite**	**CH**	Tap water	**CD**	**2**	**95mu**	**6**	**5.98**	**1.2**	**D6S+8000**	**Internal**	**30/29/30/30/30**	**5.2**	**30**	**3.0**	**0.58**	**70**	**58**	
15	KM_0_20	Monterey #30	SP	Tap water	CD	2	5/95mu	20	21.79	1.1	D6S+32000	Internal	23/23(29/29/29)	19.1	29	10.6	0.55	27	27	brittle failure
16	KM_10_20	Kaol+Mon #30	ML	Tap water	CD	2	5/95mu	20	21.79	1.1	D6S+32000	Comb./Internal	15/15/15(22/24/25/24/24)	19.8	24.5	9.0	0.46	10	18(17)	brittle failure, see pic of failure plane.(fail right after tilting to 25deg.)
17	KM_30_20	Kaol+Mon #30	CL	Tap water	CD	2	95mu	20	21.79	1.1	D6S+32000	Internal	24/25/24/24/24	19.9	24	8.9	0.45	20	16	fail right after tilting to 25deg.
18	KM_50_20	Kaol+Mon #30	CL	Tap water	CD	2	95mu	20	21.79	1.1	D6S+32000	Internal	21.5/24/23/23/23	20.1	23	8.5	0.42	35	20	
19	KM_60_20	Kaol+Mon #30	CL	Tap water	CD	2	5/95mu	20	21.79	1.1	D6S+32000	Comb./Internal	20.5/21.5/18/18/18(24/24/23/23/23)	20.1	23	8.5	0.42	42	29(25)	
20	KM_70_20	Kaol+Mon #30	CI	Tap water	CD	2	5/95mu	20	21.79	1.1	D6S+32000	Internal	15/20/20/20(22/23/23/23)	20.2	22	8.2	0.40	50	49(32)	
21	**KM_100_20**	**Kaolinite**	**CH**	Tap water	**CD**	**2**	**5/95mu**	**20**	**21.79**	**1.1**	**D6S+32000**	**Internal**	**19/20/20(19/24/24/24)**	**19.9**	**24**	**8.9**	**0.45**	**70**	**49**	

Appendix B: Raw data of Ottawa Sand

NO.	Test ID	Soil Source	Soil Classificatio	Pore water	Test type	Thickness (mm)	Interface	N stress at horizontal(kPa)	Surcharge Weight	Initial w(%)	N stress at failure	Residual secant friction angle	S stress at failure (kPa)	OCR	Coefficient of
1	S_0	Ottawa Sand	SP	Tap water	CD	2	GT	0.25	D6A+250	27	0.2	32	0.1	1.2	0.6
2	S_2	Ottawa Sand	SP	Tap water	CD	2	GT	2.01	D6S+2000	27	1.7	32	1.1	1.2	0.6
3	S_4	Ottawa Sand	SP	Tap water	CD	2	GT	4.01	D6S+5000	27	3.4	31.5	2.1	1.2	0.6
4	S_6	Ottawa Sand	SP	Tap water	CD	2	GT	5.98	D6S+8000	27	5.1	31	3.1	1.2	0.6
5	S_10	Ottawa Sand	SP	Tap water	CD	2	GT	11.25	D6S+16000	27	9.6	31	5.8	1.2	0.6
6	S_20	Ottawa Sand	SP	Tap water	CD	3	GT	21.79	D6S+32000	27	18.9	30	10.9	1.2	0.6



Appendix C: Repeatability of Test Results

In an effort to investigate the repeatability of test results, nine tests are performed on the same soil specimen. Three effective normal stresses (2, 4, and 6 kPa) are used for comparing the test results. The results of repeated tests are summarized in Table below. The differences between comparable results (i.e. same normal stress) are all within ± 1 degree. (ASTM D4318 2005)

Test ID	Interface	Soil source	Soil Type	N Stress at horizontal(kPa)	Failure type	Residual Secant Friction Angle
BC1_2	GT	BC1	ME	2.01	Internal	38
BC1_2a	GT	BC1	ME	2.01	Internal	37
BC1_2b	GT	BC1	ME	2.01	Internal	38
BC2_6	5μm	BC2	ME	5.98	Interface	35
BC2_6a	5μm	BC2	ME	5.98	Interface	35.5
BC2_6b	5μm	BC2	ME	5.98	Interface	35
BC3_4	GT	BC3	CE	4.01	Internal	39
BC3_4a	GT	BC3	CE	4.01	Internal	38.5
BC3_4b	GT	BC3	CE	4.01	Internal	39

Appendix D: Pictures for Each Failure Mechanism

<table>
<tr><th>Internal / GT / Marine Clay</th><th>Internal / Rough / Marine Clay</th></tr>
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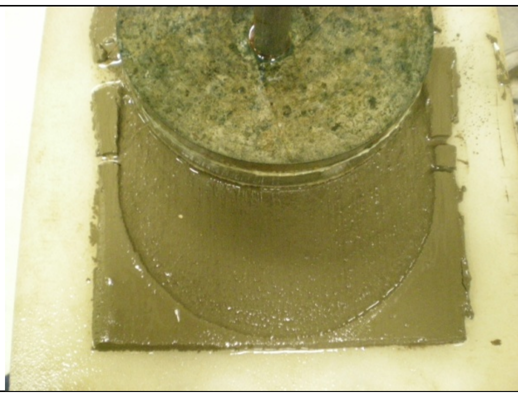
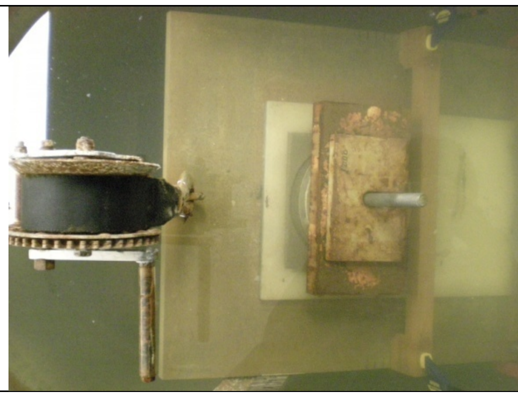
<tr><td></td><td></td></tr>
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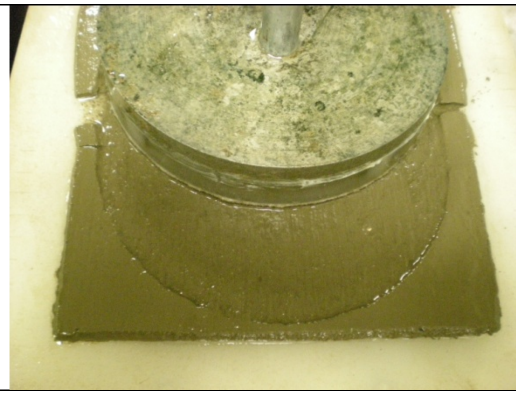
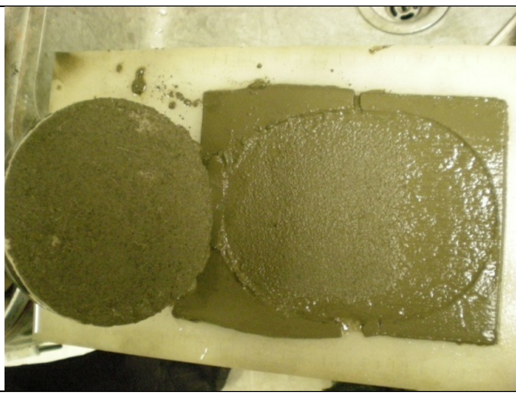
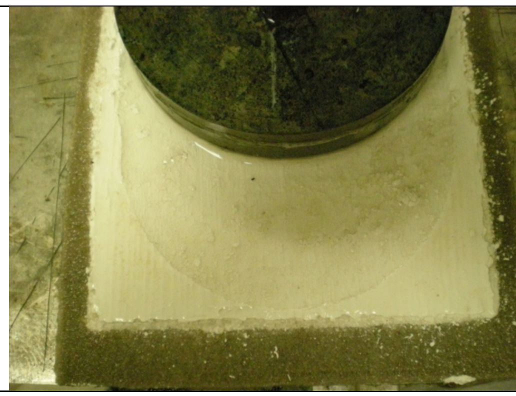
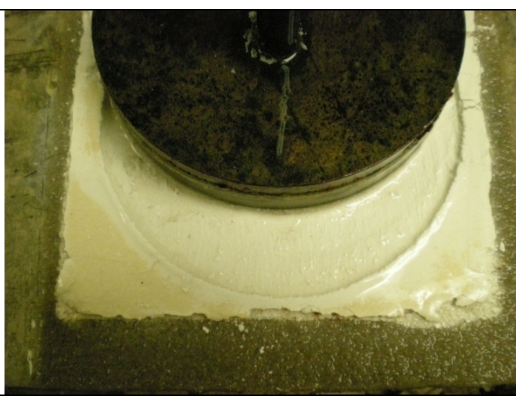
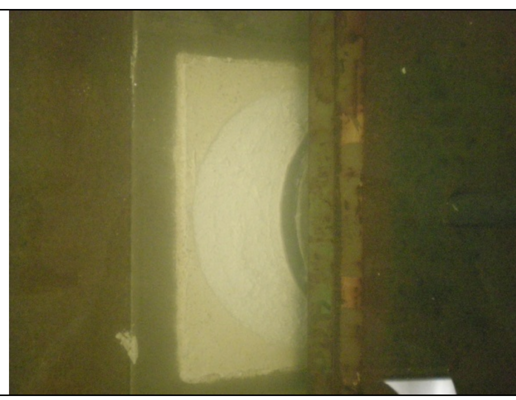
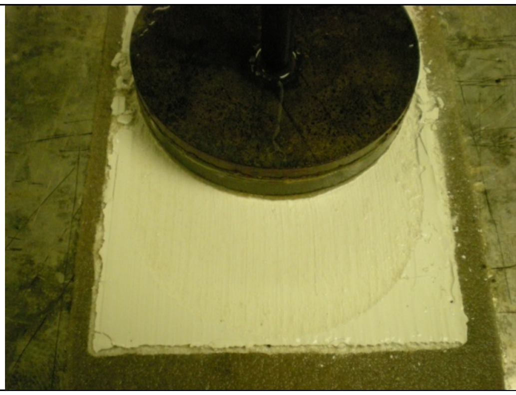
Internal / Rough / Marine Clay	Internal / Rough / Marine Clay
	

Internal / Rough / Marine Clay	Internal / GT / Kaolinite
	

Internal / GT / Kaolinite	Internal / GT / Kaolinite
	

Internal / GT / Kaolinite	Internal / GT / Kaolinite
	

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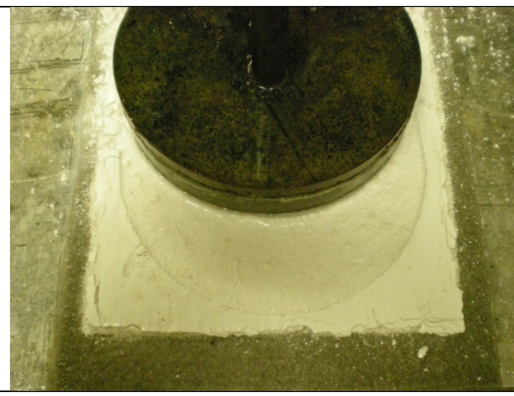
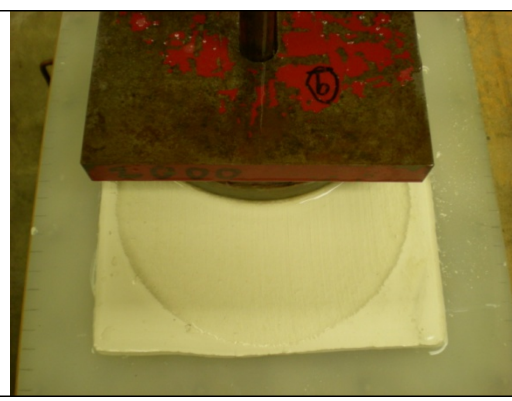
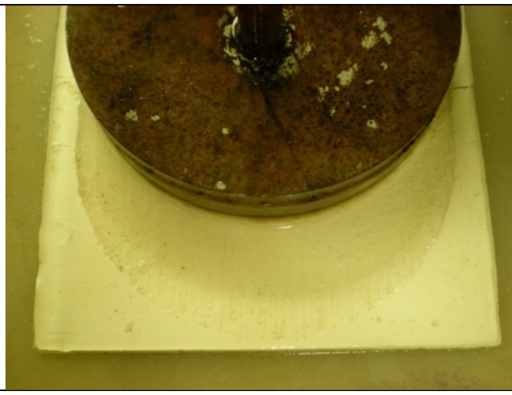
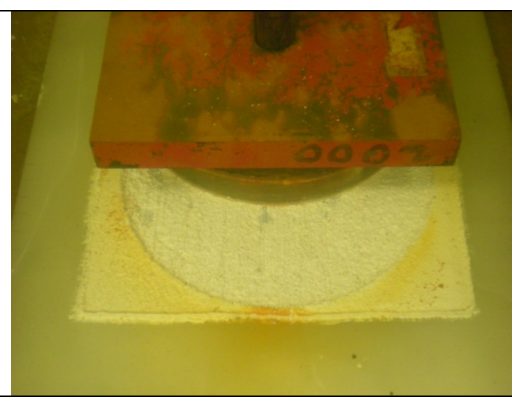
	

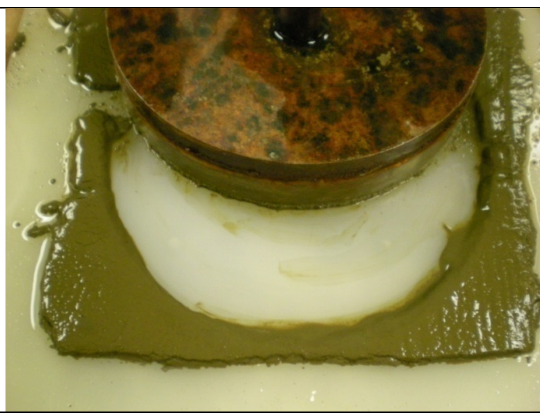
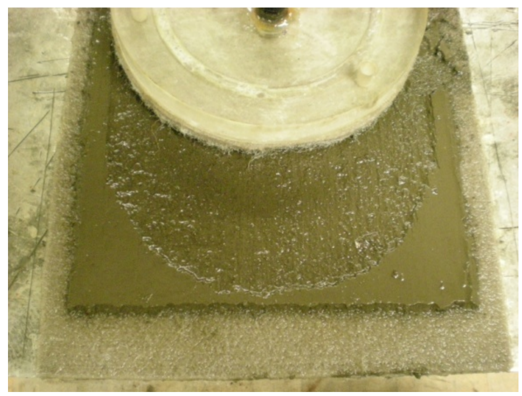
Combination / Smooth / Mixture	Internal / Smooth / Kaolinite
	

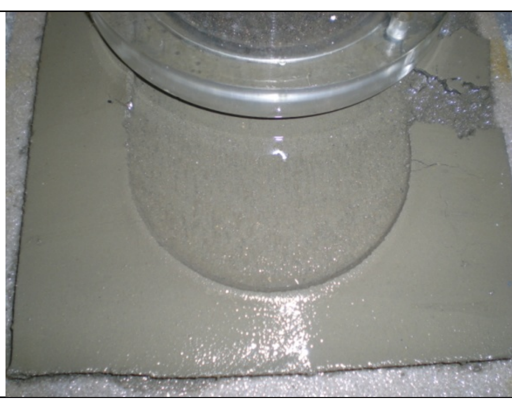
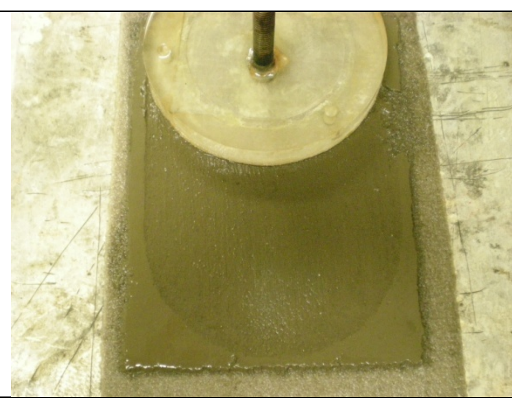
Internal / Rough / Kaolinite	Internal / Smooth / Mixture
	

Combination / Smooth / Mixture	Combination / Smooth / Mixture
	

Combination / Smooth / Mixture	Combination / Smooth / Mixture
	

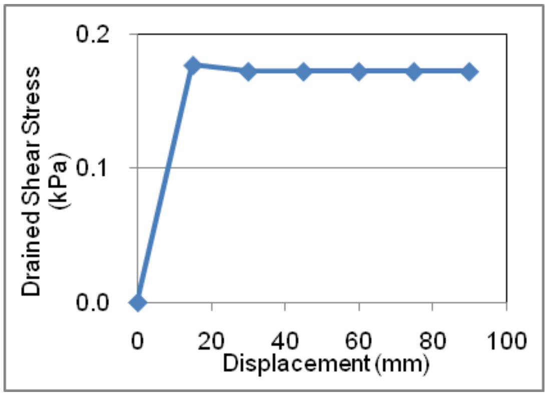
Internal / GT / Sand	Combination / Smooth / Mixture
	

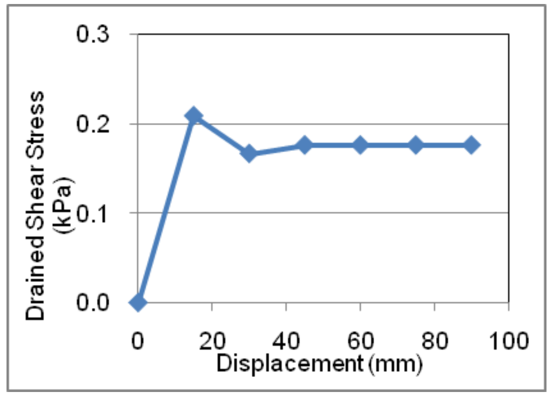
Interface / Smooth / Marine Clay	Repeat test using rough geotextile
	

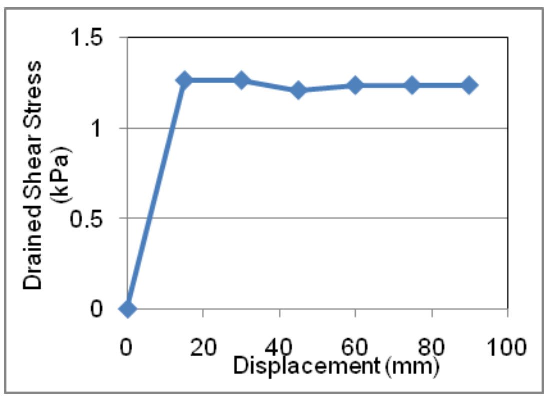
Repeat test using porous stone	Repeat test using smooth geotextile
	

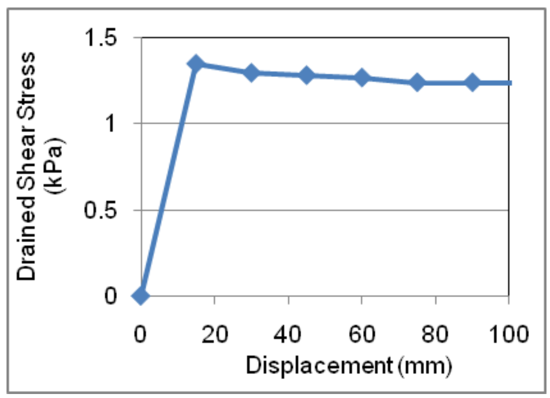
Appendix E: Load versus Displacement Curves

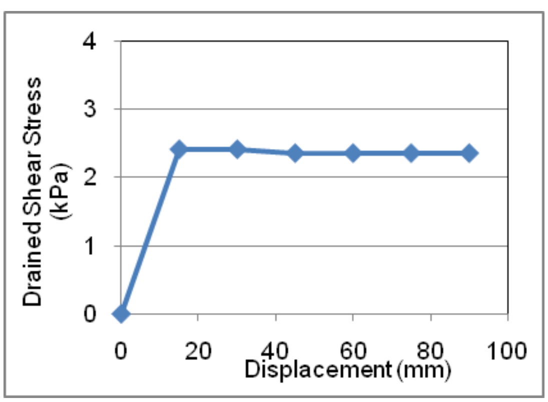
1. Marine Clays

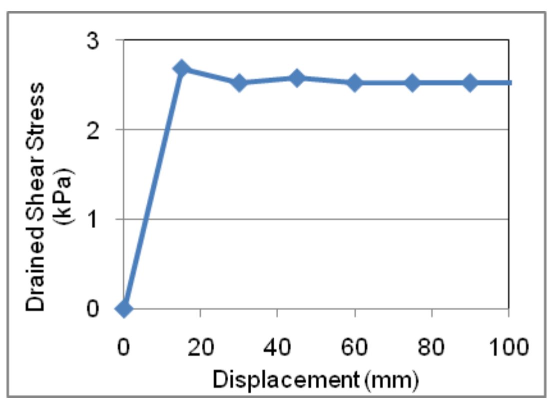


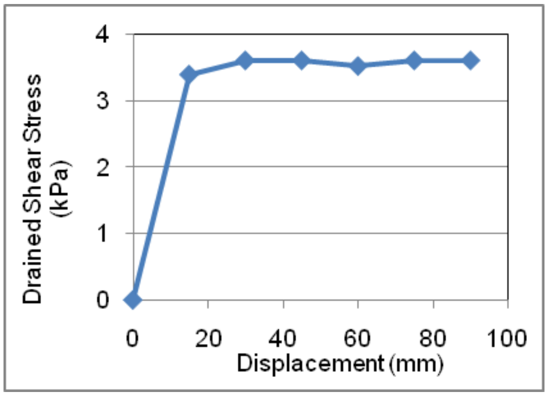








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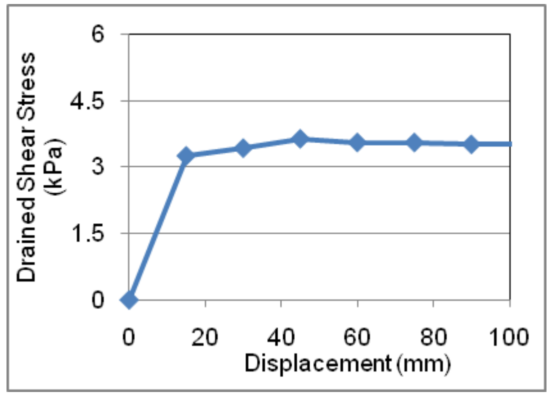
0 1 2 3 4
0 20 40 60 80 100
Displacement (mm)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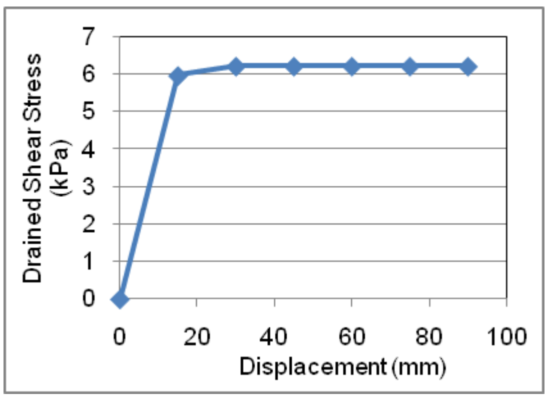
Displacement (mm)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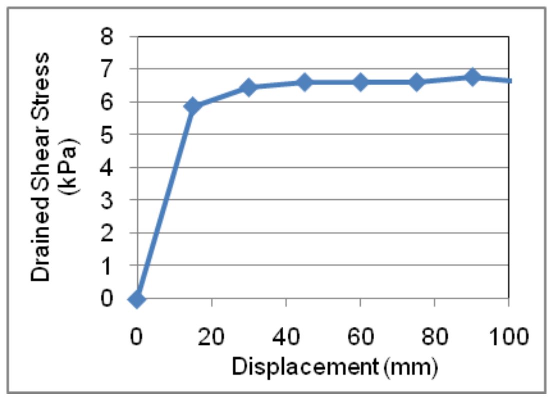
0 20 40 60 80 100
Displacement (mm)


Drained Shear Stress (kPa)
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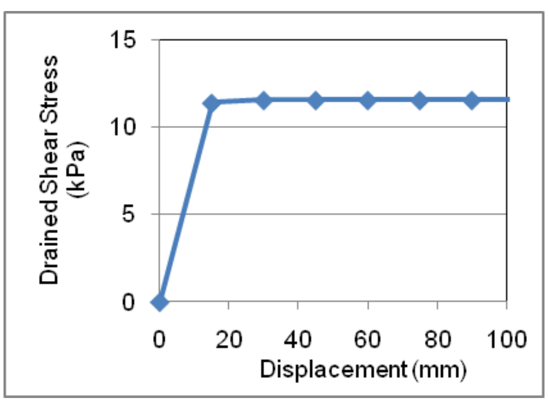
0 20 40 60 80 100
Displacement (mm)


Drained Shear Stress (kPa)
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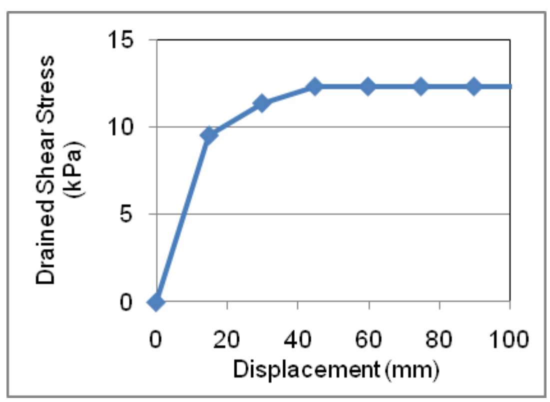
0 20 40 60 80 100
Displacement (mm)


Drained Shear Stress (kPa)
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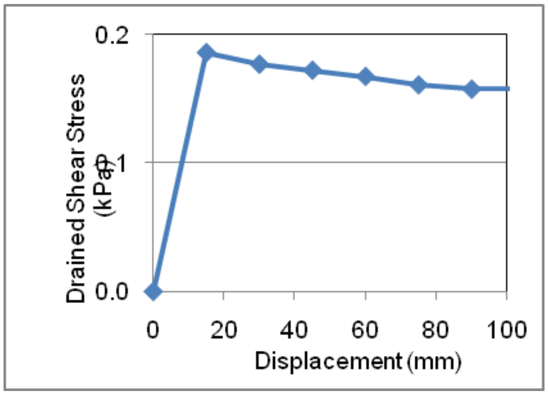
0 20 40 60 80 100
Displacement (mm)

2. Kaolinite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0.1
0.0
Drained Shear Stress (kPa)
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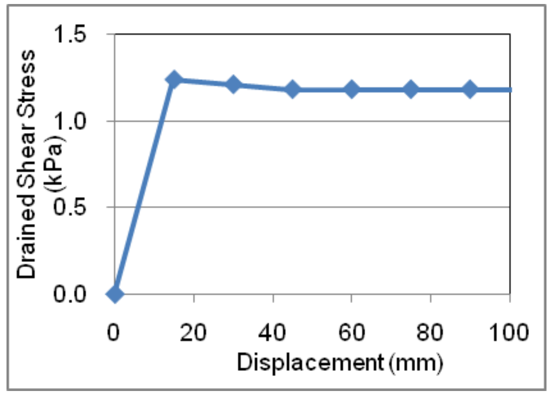
20
40
60
80
100
Displacement (mm)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0.5
0.0
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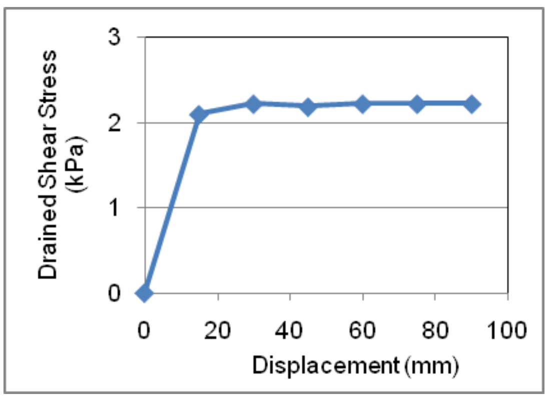
20
40
60
80
100
Displacement (mm)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2
1
0
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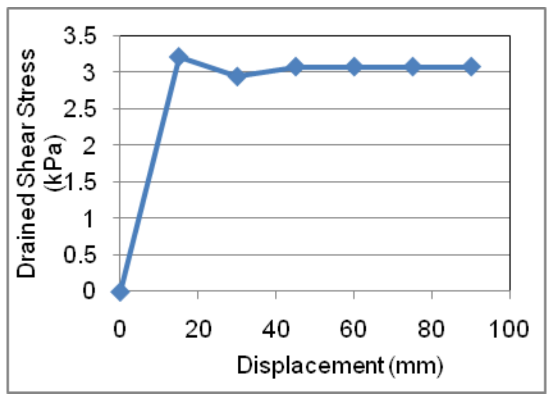
20
40
60
80
100
Displacement (mm)


3.5
3
2.5
2
1.5
1
0.5
0
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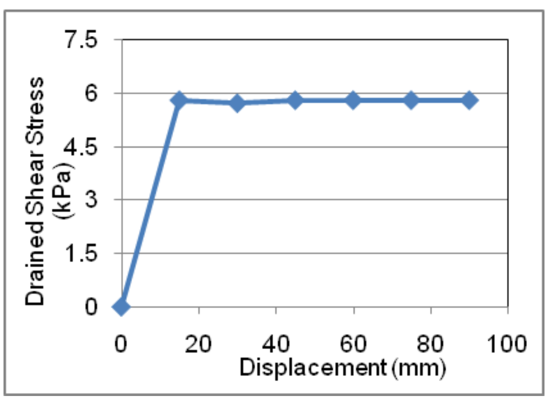
20
40
60
80
100
Displacement (mm)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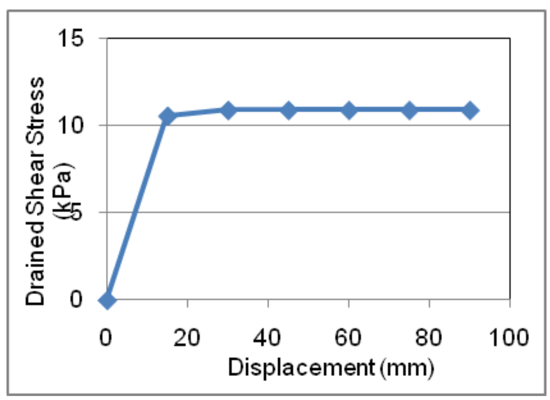
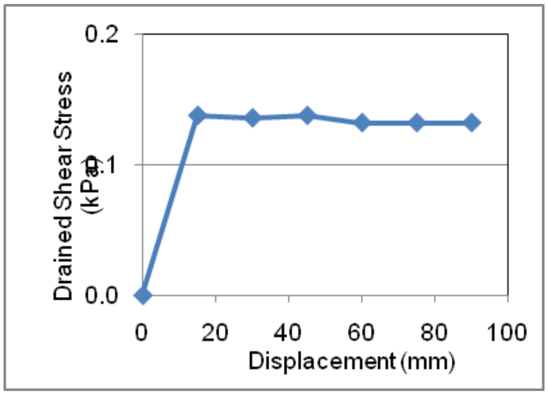
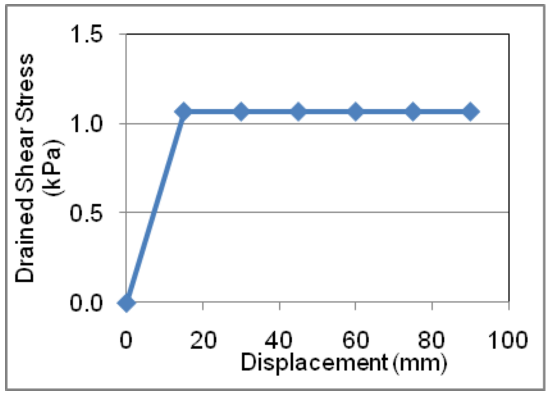
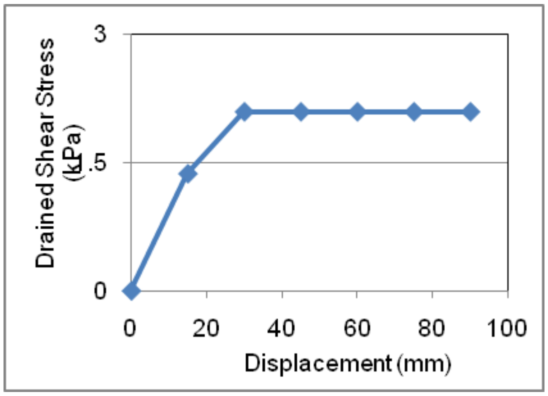
6
4.5
3
1.5
0
Drained Shear Stress (kPa)
0
20
40
60
80
100
Displacement (mm)

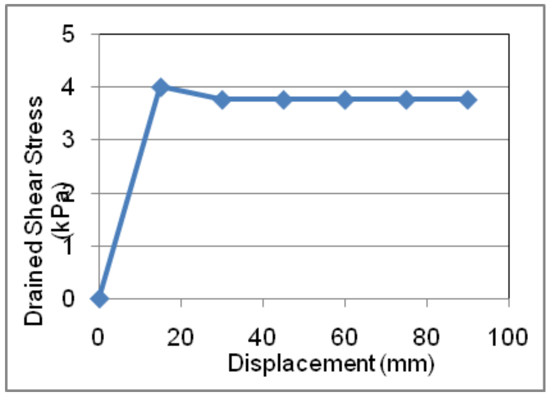

15
10
5
0
Drained Shear Stress (kPa)
0
20
40
60
80
100
Displacement (mm)

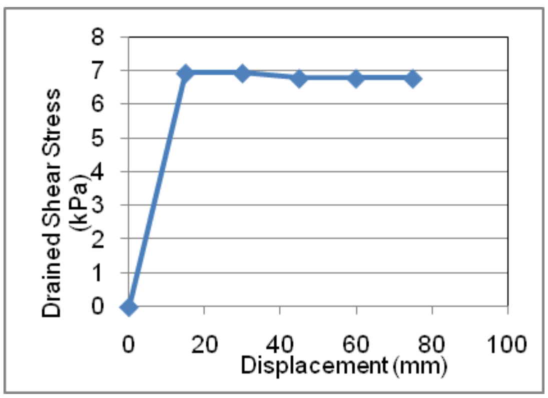
3. Sand

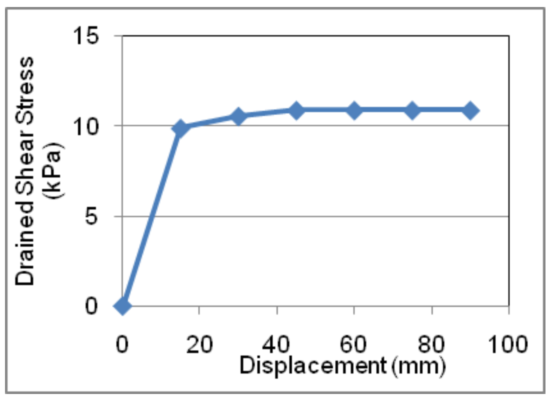

Drained Shear Stress (kPa)
0.2
0.0
0
20
40
60
80
100
Displacement (mm)


Drained Shear Stress (kPa)
1.5
1.0
0.5
0.0
0
20
40
60
80
100
Displacement (mm)


Drained Shear Stress (kPa)
3
0
0
20
40
60
80
100
Displacement (mm)


Drained Shear Stress (kPa)
5
4
3
1
0
0
20
40
60
80
100
Displacement (mm)


Drained Shear Stress (kPa)
8
7
6
5
4
3
2
1
0
0
20
40
60
80
100
Displacement (mm)


Drained Shear Stress (kPa)
15
10
5
0
0
20
40
60
80
100
Displacement (mm)

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